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# DISCUSSIONS


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NOTE.—The closing dates herein published, are final except when names of prospective discussers are registered for special extension of time.

AMERICAN SOCIETY OF CIVIL ENGINEERS

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PAPERS

# POLLUTION OF BOSTON HARBOR

By Arthur D. Weston[1] and Gail P. Edwards,[2]
Members, Am. Soc. C. E.

## Synopsis

The sewerage systems serving Metropolitan Boston are among the oldest in the United States, and the disposal works used in connection with these systems represent methods which require modifications to meet public demand for improved conditions. The discharge of sewage into Boston Harbor from these three systems has never resulted in depletion of oxygen in the tidal waters sufficient to cause a nuisance from odors but there have been increasing esthetic demands from the population residing in the vicinity of Boston Harbor for improved conditions because of sleek areas and pollution of beaches by floating matter. These increasing demands for improvements have been made regardless of the fact that the quantity of sewage discharged has not greatly increased in recent years.

The paper is framed chiefly around the results of a legislative investigation made in the years 1935 and 1936 under the general direction of the authors. It includes descriptions of the existing works and conditions of sewage disposal in harbor waters, presents results of chemical analyses and bacterial examinations sufficient in detail to permit comparison with data relative to the disposal of sewage in other tidal waters, discusses experimental work undertaken, and presents conclusions as to existing conditions and the best methods of meeting the demands for improvement.

## Introduction

Under the provisions of Chapter 42 of the Resolves of the Massachusetts Legislature of 1935, a special commission[3] was appointed to investigate the discharge of sewage into Boston Harbor and to consider what change, if any,

Note.—Written comments are invited for immediate publication; to insure publication the last discussion should be submitted by May 15, 1939.

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[3] See "Report of the Special Commission on the Investigation of the Discharge of Sewage into Boston Harbor and Its Tributaries," House Doc. No. 1600, December, 1936.

might be necessary in any of the present systems of sewerage or sewage disposal in the territory tributary to the harbor in order to prevent nuisances or to remove objectionable conditions. The investigation was conducted during 1935 and 1936.

Boston Harbor is usually defined as that body of water inside or west of a line drawn from Deer Island to Pemberton in the town of Hull and extending to the lowest bridge on each of the tributary rivers and estuaries. It consists of four bays, Dorchester, Winthrop, Quincy, and Hingham, and an inner and outer harbor. The Mystic, Charles, and Chelsea rivers and Fort Point and Reserve channels are estuaries of the inner harbor. The total area of Boston Harbor is about 27 700 acres or 43.3 sq miles at high water. The volumes of the tidal prism of the average tide which enters and leaves Boston Harbor is 76 086 gal. The quantity of water remaining at mean low tide is 108 305 million gal, and the total quantity of water at mean high tide is 184 391 million gal.

*Drainage Area of Boston Harbor.*—The tributary water-shed to Boston Harbor is about 645 sq miles. Included in this area, in whole or in part, are 53 cities and towns having a total population, according to the State Census of 1935, of 1 957 311.

Metropolitan Boston is served by three main sewerage systems (see Fig. 1), the Boston Main Drainage System, the North Metropolitan System and the South Metropolitan System. Included in this area are 32 towns and cities having a total population of 1 872 400 (or 95% of the population of the drainage area of Boston Harbor). The population contributing sewage in 1936 was estimated at 1 800 000.

*The Boston Main Drainage System.*—The Boston Main Drainage System was constructed during the period from 1877 to 1884 to intercept the sewage discharged at that time along the water-front from some 70 sewer outlets. It was designed to provide ultimately for sewage from 20 sq miles. The system was extended in 1892 and 1897 to intercept sewage from Waltham, Newton, Watertown, Brookline, the Brighton and Hyde Park section of Boston, Dedham, and Milton. Upon completion of the high-level sewer of the South Metropolitan Sewerage District in 1904 the system served only an area of 18.7 sq miles, consisting of Boston proper, South Boston, parts of Back Bay, the Roxbury, Dorchester, and West Roxbury sections of Boston, and an area of Squantum and Milton.

The system was designed to care for sewage at an average rate of 75, and a maximum rate of 112.5, gal per capita per day from a population of 800 000. In addition, it was designed for about 0.25 in. of rainfall upon the 20 sq miles in 24 hr, which was assumed to be at the rate of 0.01 in. per hr. All computations were based on the sewers running about half full and for the mingled storm water and sewage, in excess of the capacity of the sewer, to discharge through the old sewers as overflows. The resident population served by the district probably does not exceed 493 000 persons, but the per capita daily rate of sewage flow during the six months from June to November, in the period 1932 to 1935, inclusive, was 154 gal per capita instead of 75 gal per capita as originally contemplated. Estimates of flow on the basis of permanent popula-

tion may be misleading in a municipality such as Boston, which has a large transient population in manufacturing and mercantile establishments, offices, and schools. In 1927 the number of people entering and leaving the business sections each day was estimated to be 825 000.

The trunk sewer of this system has a maximum diameter of 10.5 ft and terminates at a pumping station at Calf Pasture or Old Harbor Point where, after passing through cage screens of iron bars set with an opening of 1 in., the sewage is lifted about 35 ft and is discharged through force mains into two deposit

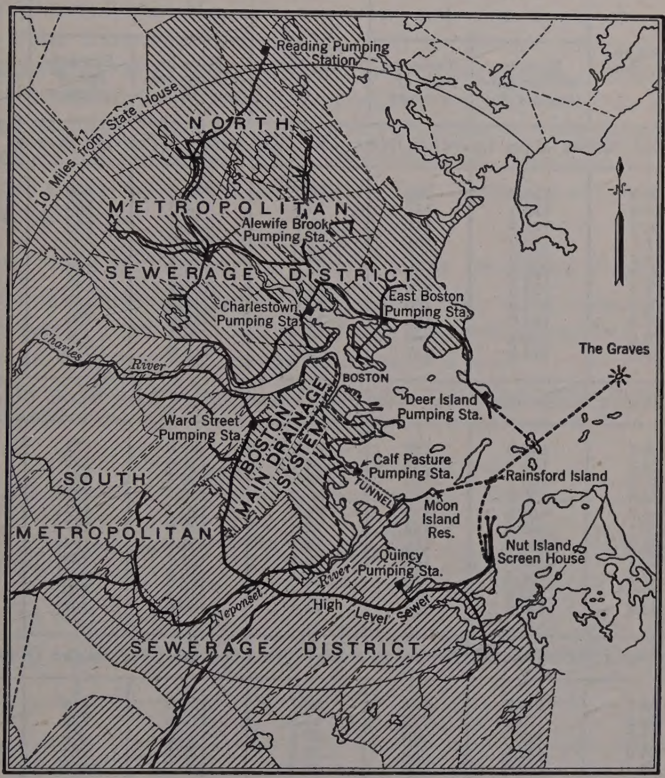


Fig. 1.—Sewerage Systems in Metropolitan Boston

sewers, each 8 ft wide, 16 ft deep, and having a total length of 1 260 ft. The velocity in these deposit sewers is about 1 ft per sec, and at their lower end the sewage flows over a weir into a tunnel to Moon Island off Squantum. The tunnel (see Fig. 1) is 7 160 ft long and 7.5 ft in diameter and is located about 143 ft below mean low water. At Moon Island the sewage is stored in four large reservoirs having a total effective capacity of approximately 35 400 000 gal. As far as practicable, sewage is discharged from the storage tanks on the second and third hours of the outgoing tide, and it enters the harbor at mean low water. The outfall sewer provided for this purpose is divided into two parts and is 10 ft 10 in. high and 12 ft wide on the inside. It extends 600 ft from the tanks to the tide water where there is a depth at mean low water of approximately 5 ft.

Table 1 shows the quantity of sewage pumped at the Calf Pasture Pumping Station and represents the quantity discharged into Boston Harbor from the Boston Main Drainage System. It is interesting to note that, whereas the

TABLE 1.—ESTIMATED QUANTITY OF SEWAGE PUMPED IN MILLION GALLONS PER DAY

| Year | For Entire Year | | | June to November, Inclusive | | | Ratio; dry weather average to daily average (percentages) |
|---|---|---|---|---|---|---|---|
| | Average daily flow | Maximum Day | | Average daily flow | Maximum Day | | |
| | | Flow in million gallons per day | Percentage of average | | Flow in million gallons per day | Percentage of average | |
| **(a) Boston Main Drainage System; at Calf Pasture Pumping Station** | | | | | | | |
| 1925 | 76.2 | 125.8 | 165 | 77.7 | 115.8 | 150 | 102 |
| 1926 | 80.3 | 126.0 | 157 | 76.0 | 113.5 | 150 | 95 |
| 1927 | 86.6 | 121.5 | 141 | 87.0 | 121.3 | 140 | 101 |
| 1928 | 71.7 | 113.0 | 157 | 72.6 | 111.9 | 155 | 101 |
| 1929 | 76.8 | 157.0 | 205 | 73.2 | 120.0 | 164 | 95 |
| 1930 | 79.0 | 133.5 | 169 | 84.0 | 133.1 | 159 | 106 |
| 1931 | 78.3 | 133.5 | 170 | 74.0 | 133.1 | 180 | 94 |
| 1932* | 76.0 | 125.8 | 166 | 77.3 | 118.0 | 153 | 102 |
| 1933* | 77.8 | 144.0 | 186 | 74.8 | 144.0 | 193 | 96 |
| 1934* | 79.4 | 129.1 | 163 | 79.8 | 129.1 | 162 | 100 |
| 1935* | 75.3 | 118.0 | 157 | 72.3 | 118.0 | 164 | 96 |
| **(b) North Metropolitan Sewerage System through the Deer Island Outlet** | | | | | | | |
| 1925 | 78.1 | 148.0 | 189 | 70.9 | 121.0 | 171 | 91 |
| 1926 | 79.3 | 143.8 | 181 | 74.1 | 131.0 | 177 | 94 |
| 1927 | 84.0 | 150.7 | 180 | 80.9 | 136.3 | 169 | 96 |
| 1928 | 86.9 | 150.7 | 174 | 89.1 | 150.7 | 169 | 103 |
| 1929 | 84.7 | 151.6 | 179 | 75.7 | 129.5 | 171 | 89 |
| 1930 | 77.1 | 130.4 | 169 | 72.8 | 129.0 | 177 | 94 |
| 1931 | 84.2 | 154.7 | 184 | 79.7 | 154.7 | 194 | 95 |
| 1932 | 82.4 | 151.7 | 184 | 80.1 | 151.7 | 189 | 97 |
| 1933 | 82.1 | 151.5 | 185 | 72.4 | 150.2 | 208 | 88 |
| 1934 | 83.6 | 150.9 | 180 | 74.2 | 117.4 | 158 | 89 |
| 1935 | 82.9 | 150.0 | 181 | 74.4 | 146.6 | 197 | 90 |
| **(c) South Metropolitan Sewerage System; Through the Nut Island Outfall** | | | | | | | |
| 1925 | 63.7 | ‡ 166.0 | ‡ 261 | 52.7 | ‡ 166.0 | ‡ 315 | 83 |
| 1926 | 62.2 | 170.0 | 273 | 47.5 | 113.5 | 239 | 76 |
| 1927 | 68.2 | 162.0 | 238 | 63.6 | 162.0 | 254 | 93 |
| 1928 | 64.9 | 162.0 | 250 | 58.8 | 137.5 | 234 | 91 |
| 1929 | 64.4 | 199.0 | 309 | 46.0 | 129.5 | 282 | 71 |
| 1930 | 61.0 | 166.0 | 272 | 51.7 | 144.5 | 280 | 85 |
| 1931 | 76.3 | 220.5 | 289 | 66.5 | 220.5 | 332 | 87 |
| 1932 | 68.4 | 205.0 | 300 | 48.7† | 162.01† | 333 | 71 |
| 1933 | 84.1 | 227.0 | 270 | 70.8 | 227.0 | 320 | 84 |
| 1934 | 74.2 | 215.0 | 290 | 62.3 | 150.0 | 241 | 84 |
| 1935 | 86.5 | 230.0 | 266 | 70.5 | 145.0 | 206 | 82 |

* Pump slippage and other losses estimated at 20% because of new equipment instead of 30% of previous years.

† June to September, inclusive.

‡ Represent maximum rate during any period; gravity flow.

maximum daily pumpage throughout each year has varied from 141% to more than 200% of the average daily value, the average daily amount pumped during June to November, inclusive, has varied little from that pumped on the average throughout the entire year.

## Overflow of Sewage from Boston Main Drainage System

The Boston Main Drainage System was designed to take care of some surface run-off in addition to the sanitary sewage; but for a number of years it has been considerably surcharged during storms because of the much larger amount of surface drainage that actually enters the system. At present (1939), during certain storms large quantities of mingled storm water and sewage enter the Charles River Basin and cause objectionable conditions. The Special Commission recommended[3] that legislation be provided to compel separation of storm water from domestic sewage in the city of Boston and that funds be appropriated to permit the construction of adequate works for removing storm water from the sewers in this district. Because of the impracticability of separating storm water from domestic sewage in the business sections of Boston, the ultimate solution of the problem probably would be to provide relief sewers of large diameter to divert the overflow of mingled storm water and sewage to parts of Boston Harbor where the least objectionable conditions may result so that the waters of the Charles River Basin and at certain beaches may not be polluted.

## Quality of Sewage from Boston Main Drainage District

Chloride determinations indicate that about 23% of the volume of sewage pumped in recent years has consisted of salt water which entered the system through leaks, or through tide gates, or by other means.

Table 2(*a*) gives the results of the analyses of the samples collected during the 1935–1936 investigation[3] and during the investigations of 1913 and 1929.

TABLE 2.—Sewage Analyses

| Year | Number of samples | Average flow in million gallons daily | Free Ammonia | | Total Albuminoid Ammonia | | Oxygen Consumed | | Chlorides in parts per million | Five day 20° Centigrade B.O.D. in parts per million | Kjeldahl Nitrogen | | Total suspended solids in parts per million |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | | | In parts per million | In pounds per day | In parts per million | In pounds per day | In parts per million | In pounds per day | | | In parts per million | In pounds per day | |
| (*a*) Boston Main Drainage System | | | | | | | | | | | | | |
| 1913 | 24 | 67.1 | 27.9 | 15 500 | 10.0 | 5 580 | 77.4 | 43 200 | 2 226 | .. | 20.8 | 11 000 | .. |
| 1929 | 25 | 76.5 | 22.7 | 14 450 | 4.2 | 2 680 | 34.5 | 22 000 | 4 142 | .. | .. | .. | .. |
| 1935 | 230 | 75.4 | 21.5 | 13 400 | 4.1 | 2 560 | 53.0 | 33 100 | 4 080 | 157 | 9.05 | 5 650 | 162 |
| 1936 | 72 | 86.0 | 18.1 | 12 990 | 3.4 | 2 440 | 49.6 | 35 600 | 3 478 | 144 | 8.6 | 6 170 | 119 |
| (*b*) North Metropolitan Sewerage System | | | | | | | | | | | | | |
| 1913 | 41 | 56.5 | 25.0 | 11 800 | 13.4 | 6 300 | 106.0 | 50 000 | 2 290 | .. | 25.5 | 12 000 | .. |
| 1929 | 49 | 84.7 | 25.7 | 18 050 | 6.36 | 4 470 | 86.2 | 60 600 | 3 856 | .. | .. | .. | .. |
| 1935 | 81 | 82.9 | 34.1 | 23 500 | 7.52 | 5 180 | 81.0 | 55 800 | 3 280 | 249 | 16.4 | 11 300 | 256 |
| 1936 | 105 | 86.4 | 26.2 | 18 890 | .. | .. | 69.0 | 49 750 | 3 151 | 175 | 11.4 | 8 220 | 152 |
| (*c*) South Metropolitan Sewerage System | | | | | | | | | | | | | |
| 1913 | 6 | 53.0 | 16.1 | 7 080 | 4.6 | 2 030 | 35.5 | 15 700 | 204 | .. | 9.6 | 4 200 | .. |
| 1929 | 24 | 64.4 | 21.2 | 11 350 | 3.45 | 1 845 | 30.9 | 16 550 | 134 | .. | .. | .. | .. |
| 1936 | 64 | 96.5 | 20.3 | 16 350 | 3.71 | 3 096 | 50.0 | 40 240 | 198 | 119 | 9.6 | 7 730 | 156 |

The material removed from the bar cage screens at the Calf Pasture Pumping Station is deposited on an adjacent dump to be drained and then burned.

## Adequacy of the Boston Main Drainage System

At times of heavy run-off the storm water entering the main trunk sewers of the Boston Main Drainage System exceeds the capacity of the tunnel from Calf Pasture to Moon Island and the capacity at which it is practicable to operate the pumps at the Calf Pasture Pumping Station. At such times, large quantities of mingled storm water and sewage must discharge through the overflows provided for the purpose. If the full capacity of this main drainage system were utilized by increasing the pumping, the system could be made to deliver about 244 million gal in 24 hr; but this quantity is about 56% in excess of the designed capacity of the tunnel from Calf Pasture to Moon Island. Moreover, if it were possible to pass this quantity of sewage through the tunnel to Moon Island it would be necessary to increase the tank storage considerably at Moon Island if the sewage is to be discharged only on the second and third hours of the outgoing tide. Under present conditions these tanks are not allowed to fill above Grade 20, to prevent crowding of the tunnel and overflow of sewage from the deposit sewers at Calf Pasture.

It was the original intention that the stored sewage would be discharged only on the second and third hours of the outgoing tide, but from June to November, inclusive, during the eight years 1928 to 1935, the maximum average daily quantity of sewage pumped was about 84 million gal. Accordingly, it has been necessary, at times, to discharge the sewage on other than the second and third hours of the outgoing tide. The 1936 investigation showed that during the period from 1928 to 1936, inclusive, there have been 1 034 times when sewage was discharged either before or after the period in which the sewage was intended to be discharged, about 97 of which were made necessary by experimental work during the investigation.

## Pollution of Boston Harbor by the North Metropolitan Sewerage District

The North Metropolitan Sewerage District was created by acts of the Massachusetts Legislature in 1889 to provide sewerage facilities in the drainage areas of the Charles and Mystic rivers as recommended by the State Board of Health in the same year. The proposed Metropolitan District was to be divided into two parts, one part for the district north of the Charles River to be known as the North Metropolitan Sewerage District, and the other to provide for the diversion of sewage from the Charles River Valley into the Boston Main Drainage System. The North Metropolitan Sewerage District was to include the Charlestown and East Boston sections of Boston and the cities of Cambridge, Somerville, Malden, Chelsea, Medford, Melrose, Everett and Woburn, and the towns of Stoneham, Winchester, Arlington, Belmont, and Winthrop. Additional legislation has been provided from time to time authorizing extensions of the North Metropolitan Sewerage District, and in 1935 the size of the system and district[4] was substantially as shown in Table 3.

[4] "Report of the Special Commission on the Investigation of the Discharge of Sewage into Boston Harbor and Its Tributaries," House Doc. No. 1600, December, 1936, p. 95.

TABLE 3.—AREAS AND POPULATIONS SERVED BY VARIOUS SEWERAGE DISTRICTS

| Cities and towns | Miles of local sewers connected | Number of connections with local sewers | Estimated number of persons served by each house connection | Estimated population contributing sewage in 1936 | Estimated present total population | Estimated area contributing sewage in 1936 in square miles | Area ultimately to contribute sewage in 1936 in square miles | Ratio of contributing population to present total population (percentages) | Ratio of contributing area to ultimate area (percentages) |
|---|---|---|---|---|---|---|---|---|---|
| (1) | (2) | (3) | (4) | (5) | (6) | (7) | (8) | (9) | (10) |
| *(a)* NORTH METROPOLITAN SEWERAGE SYSTEM (Populations Estimated as of December 31, 1935) | | | | | | | | | |
| Arlington....... | 64.56† | 6 329 | 5.60 | 35 440 | 39 060 | 3.02 | 5.20 | 90.7 | 58.1 |
| Belmont........ | 51.26† | 3 628 | 7.00 | 25 400§ | 26 280 | 2.40 | 4.66 | 96.7 | 51.5 |
| Boston: | | | | | | | | | |
| Deer Island... | 0.70† | .... | .... | 930‖ | 930 | .... | .... | .... | .... |
| East Boston... | 35.96‡ | 5 549 | 11.20 | 62 150 | 64 870 | 1.27 | 2.18 | 95.8 | 58.3 |
| Charlestown... | 22.04‡ | 5 614 | 5.18 | 29 080 | 29 200 | 0.67 | 1.27 | 99.6 | 52.8 |
| Cambridge...... | 165.17‡ | 19 213 | 6.17 | 118 540 | 119 000 | 5.18 | 6.11 | 99.6 | 84.8 |
| Chelsea......... | 33.04‡ | 4 865 | 8.50 | 41 350 | 42 010 | 1.23 | 2.24 | 98.4 | 54.9 |
| Everett......... | 53.46‡ | 6 698 | 6.90 | 46 220 | 46 980 | 2.15 | 3.34 | 98.4 | 64.4 |
| Lexington....... | 17.44† | 801 | 4.30 | 3 440 | 11 090 | 0.97 | 16.20 | 31.0 | 6.0 |
| Malden......... | 81.41† | 9 581 | 5.90 | 56 530 | 57 120 | 3.59 | 5.07 | 99.0 | 70.8 |
| Medford......... | 97.16† | 10 975 | 5.60 | 61 460 | 61 800 | 4.47 | 8.35 | 99.4 | 53.5 |
| Melrose......... | 51.89† | 5 175 | 4.70 | 24 320 | 24 490 | 2.29 | 3.73 | 99.3 | 61.4 |
| Reading........ | 11.15† | 539 | 4.20 | 2 260 | 10 900 | 0.54 | 9.82 | 20.7 | 5.5 |
| Revere......... | 54.50† | 5 403 | 6.30 | 34 040 | 35 250 | 2.54 | 5.86 | 96.6 | 43.3 |
| Somerville...... | 106.78‡ | 17 976 | 5.55 | 99 770 | 100 110 | 3.68 | 3.96 | 99.7 | 92.9 |
| Stoneham....... | 20.74† | 1 616 | 4.70 | 7 600 | 11 000 | 1.04 | 5.50 | 69.1 | 18.9 |
| Wakefield....... | 27.06† | 1 784 | 4.50 | 8 030 | 16 530 | 1.15 | 7.65 | 48.6 | 15.0 |
| Winchester...... | 43.12† | 3 025 | 4.40 | 13 310 | 13 510 | 2.02 | 5.95 | 98.5 | 33.9 |
| Winthrop....... | 33.81† | 3 896 | 4.35 | 16 950 | 17 030 | 1.43 | 1.61 | 99.5 | 88.8 |
| Woburn......... | 28.27† | 1 755 | 5.50 | 9 650 | 19 750 | 1.35 | 12.71 | 48.9 | 10.6 |
| Totals...... | 999.52 | 114 422 | 6.10 | 696 470 | 746 910 | 40.99 | 111.41 | 93.2 | 36.8 |
| *(b)* SOUTH METROPOLITAN SEWERAGE SYSTEM | | | | | | | | | |
| Boston: | | | | | | | | | |
| West Roxbury | 98.41‡ | 7 904 | 8.30 | 65 600¶** | 86 230¶ | 3.86 | 8.92 | 76.1 | 43.3 |
| Roxbury*..... | .... | .... | .... | .... | 60 500¶ | .... | 1.23 | .... | .... |
| Back Bay..... | 27.84‡ | 2 251 | 22.20 | 49 970 | 50 190 | 1.17 | 1.61 | 99.6 | 72.7 |
| Brighton...... | 74.85‡ | 6 024 | 11.65 | 70 180 | 70 360 | 3.41 | 3.74 | 99.7 | 91.2 |
| Dorchester.... | 74.00‡ | 8 414 | 12.50 | 105 175¶ | 150 140¶ | 2.97 | 4.89 | 70.1 | 60.7 |
| Hyde Park.... | 44.85† | 3 475 | 9.60 | 33 360 | 33 620 | 1.99 | 4.57 | 99.2 | 43.5 |
| Braintree....... | 16.23† | 167 | 4.30 | 720 | 17 410 | 0.76 | 13.44 | 4.1 | 5.6 |
| Brookline....... | 94.71‡ | 7 423 | 6.80 | 50 480 | 50 910 | 4.28 | 6.81 | 99.2 | 62.8 |
| Canton......... | 3.11† | 275 | 4.90 | 1 350 | 6 650 | 0.15 | 17.84 | 20.3 | 0.8 |
| Dedham........ | 23.56† | 1 494 | 4.65 | 6 950 | 15 420 | 1.13 | 9.66 | 45.1 | 11.7 |
| Milton.......... | 35.69‡ | 2 817 | 4.75 | 13 380¶ | 18 510¶ | 1.55 | 12.59 | 72.3 | 12.3 |
| Needham....... | 18.55† | 696 | 4.80 | 3 340 | 12 040 | 0.87 | 12.50 | 27.7 | 7.0 |
| Newton......... | 188.17† | 13 246 | 4.95 | 65 570 | 66 320 | 9.41 | 16.88 | 98.9 | 55.7 |
| Norwood........ | 32.10† | 2 141 | 6.10 | 13 060 | 15 680 | 1.70 | 10.16 | 83.3 | 16.7 |
| Quincy......... | 151.87† | 13 176 | 5.85 | 77 080 | 77 950 | 5.82 | 12.56 | 98.9 | 46.3 |
| Stoughton....... | .... † | .... | .... | .... | 8 540 | .... | 16.23 | .... | .... |
| Walpole........ | 5.79† | 53 | 4.70 | 1 515 | 7 490 | 0.18 | 20.54 | 20.2 | 0.9 |
| Waltham........ | 64.89† | 5 380 | 7.90 | 42 500†† | 42 920†† | 3.62 | 13.63 | 99.0 | 26.6 |
| Watertown...... | 63.56† | 6 096 | 5.80 | 35 360 | 36 020 | 2.95 | 4.04 | 98.2 | 73.0 |
| Wellesley....... | 38.83† | 1 737 | 5.90 | 8 100 | 13 790 | 2.17 | 9.89 | 58.7 | 21.9 |
| Weymouth...... | .... † | .... | .... | .... | 21 930 | .... | 16.46 | .... | .... |
| Totals...... | 1 057.01 | 82 769 | 7.8 | 643 690 | 862 620 | 47.99 | 218.19 | 74.6 | 22.0 |

* At present connected with Boston Main Drainage System.

† Separate. ‡ Separate and combined. § Including two connections with McLean Hospital, having an estimated population of 803. ‖ Estimated by Superintendent of the Institution on Deer Island.

¶ Parts of Dorchester, West Roxbury, Roxbury and Milton, which are situated within the South Metropolitan Sewerage District limits, are tributary to the Boston Main Drainage System.

** Including connection with the Boston State Hospital, having an estimated population of 2 873.

†† Including connections with the Metropolitan State Hospital and the Middlesex County Tuberculosis Hospital, authorized by Chapter 372 of the Acts of 1928 and Chapter 373 of the Acts of 1929, having an estimated population of 2 078.

The North Metropolitan Sewerage System was designed to exclude storm water except in the cities of Cambridge and Somerville; and, to provide for an ultimate population of 571 000 in 1930, and for a flow of 210 cu ft per sec, or about 136 million gal per day. It was assumed that the volume of sewage to be cared for would be 30 cu ft per person in 24 hr except in the cities of Cambridge and Somerville where the sewers are on the combined plan and where an allowance of 35 cu ft per person per 24 hr was made.

There are approximately 74 miles of Metropolitan sewers in the North Metropolitan Sewerage District which vary in diameter from 10 to 109 in. The main sewer terminates at a pumping station at Deer Island where the sewage is screened and, by means of low-lift pumps, raised to an elevation that will permit it to discharge into the harbor by gravity through a system of multiple outlets which extends from the Deer Island Pumping Station to the vicinity of Deer Island Light. The discharge of sewage through the original outlet at this site caused complaints from the keeper of the Deer Island Light, and in 1917 the multiple outlets were provided. The present outfall, at its lower end, consists of 9-ft lengths of cast-iron pipe varying in diameter from 84 in. to 48 in. This pipe is open at the end and the thirteen lengths of pipe near the end have openings on the top. It is through these thirteen openings and the one at the end that sewage is discharged. The thirteen openings are practically elliptical in cross-section and vary in size from 44 in. by 25 in. to 23 in. by 13 in. The depth of water in the vicinity of these outlets is from 30 to 50 ft.

All the sewage from the district except Winthrop is pumped at a station at East Boston before reaching the Deer Island Station. The district also provides stations to pump the sewage from a section of Charlestown and from the Alewife Brook sewer which receives sewage from sections of Cambridge, Somerville, and Belmont; and a station is provided by the town of Winthrop to pump the sewage from that town into the North Metropolitan Sewerage System. Sewage at all of the Metropolitan pumping stations is screened through wrought-iron barred cage screens provided in duplicate.

## Quantity of Sewage from North Metropolitan Sewerage District

The estimate of sewage flow for the North Metropolitan District does not include the large quantities of mingled storm water and sewage entering Boston Harbor or its tributaries through storm overflows; and, accordingly, it cannot be considered alone in determining the adequacy of the North Metropolitan Sewerage System. Fig. 2 shows the estimates of flow of sewage from the North Metropolitan Sewerage District made by the Metropolitan District Commission. Table 1(*b*), for the period 1925 to 1935, includes maximum flow as a percentage of the average daily flow. The comparison of Table 1(*b*) with Table 1(*a*) shows that the flow during the maximum day is considerably greater from the North Metropolitan Sewerage District than from the Boston Main Drainage District.

### Adequacy of the North Metropolitan Sewerage System

A comparison of the average daily flow for 1935 (82.9 mgd) with a flow of 136 mgd, which is the capacity for which the system was designed, would indicate that the capacity of this system in the maximum section is approximately 64% in excess of the average daily flow. The population served by this system in 1935 (696 470) was about 22% in excess of the 571 000 for

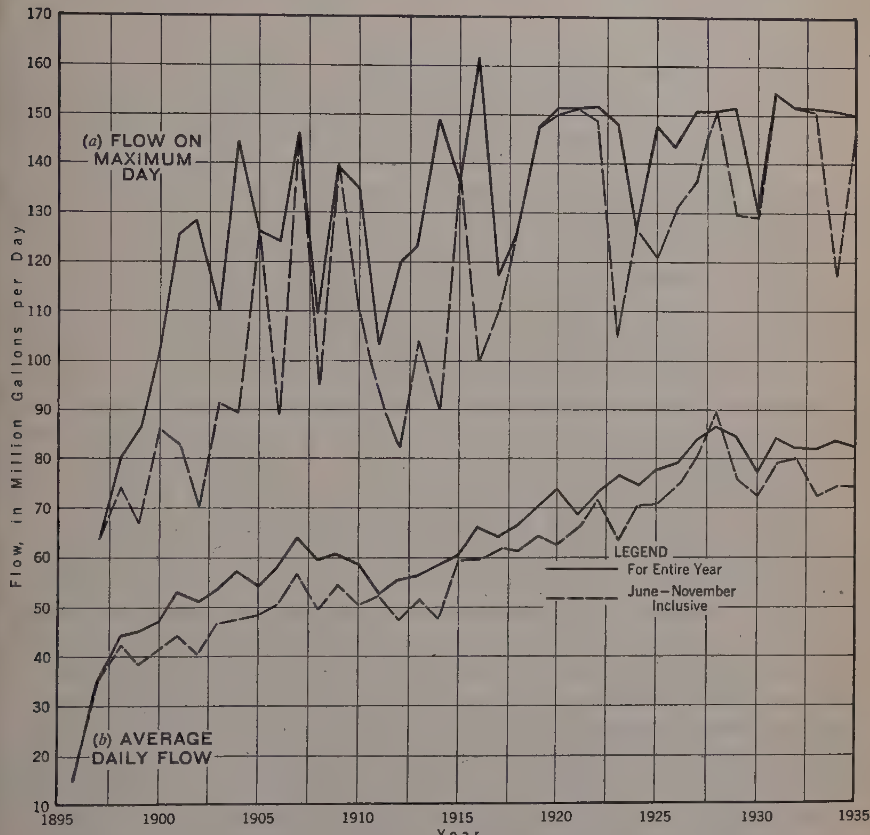


Fig. 2.—Sewage Flow, North Metropolitan System

which the system was designed. If storm water were excluded from the system except in the cities of Somerville and Cambridge, as originally proposed, many of the main trunk sewers in the lower part of the system would have adequate capacity to care for sanitary sewage. It should also be stated that, although the total population served by the system is only 22% in excess of that for which the system was designed, actually in some parts (namely, in the towns of Arlington and Belmont and Winchester) the 1935 population is from 3 to 12 times what it was when the system was designed.

## Character of the Sewage of the North Metropolitan District

Samples collected by the Massachusetts Department of Public Health during the investigations of 1929–1930 and 1935–1936 show that the sewage of the North Metropolitan Sewerage District is somewhat stronger than that of either the South Metropolitan or the Boston Main Drainage Districts. Approximately 19% of the sewage pumped at Deer Island consists of salt water which is believed to have entered the system through gates or leaks. Table 2(*b*) contains the analyses of the sewage from this district during 1913, 1929, 1935, and 1936.

Because of the large quantity of mingled storm water and sewage overflowing from the North Metropolitan System into water courses, the Metropolitan District Commission began, in 1935, an extensive program involving the construction of a relief sewer to divert this mingled storm water and sewage away from the inland streams, and the Special Commission, herein referred to, has specifically recommended the further separation of storm water from the domestic sewage in this district. It was the intent of the Special Commission to create a means of charging municipalities according to flow and hence for much of the leakage and storm water entering the main system from local sewers.

## South Metropolitan Sewerage System

The South Metropolitan Sewerage System was constructed, under the provisions of Chapter 424 of the Acts of 1899, to serve the then Charles River Valley Metropolitan Sewerage District, which comprised a part of Boston, all of Newton, Waltham, Watertown, and Brookline; and, the then Neponset Valley Metropolitan Sewerage District, which comprised a part of Boston, all of Dedham, Hyde Park, Milton, Quincy, and such parts of Dorchester, Roxbury, and West Roxbury as were not included in the then Metropolitan Sewerage areas. This system includes a high-level intercepting trunk sewer, from Parker Hill in Roxbury to Nut Island in Quincy, which varies in size from a section 68 by 72 in. to a section 135 by 150 in. The sewage from the low-lying sections tributary to the Charles River Valley Sewer and the sewage from sections of the town of Braintree and the city of Quincy are pumped into this sewer. Table 3(*b*) shows the area and population served by this district.

Three outlets are provided for the sewage from the South Metropolitan District, each of which consists of a cast-iron outfall sewer, 5 ft in diameter. The outlet through which the sewage is most commonly discharged is about a mile north of the screen house and about 1 200 ft from the westerly end of Peddocks Island. The second outlet, which is used at times in common with the first, is about the same distance from Nut Island and about 1 300 ft west of the first outlet, whereas the third outlet which is about 1 400 ft from the screen house has a point of discharge about 2 300 ft from the westerly shore of Peddocks Island. This latter outlet is used only in cases of emergency or at times of heavy run-off. At mean low tide the water over the two main outlets is about 30 ft deep and over the emergency outlet, 20 ft deep. These outlet pipes are laid in a trench at an average depth of about 9 ft below the bottom of the harbor. The outlets are encased in concrete and consist of a special

casting in the form of a quadrant bend, 11 ft long weighing about 11 tons. Each pipe is surrounded by a rectangular timber casing 13 ft sq and 10 ft deep, resting upon 12 piles, 22 ft long. These structures are surmounted and surrounded by cut granite rings in six pieces, 30 in. deep.

The South Metropolitan Sewerage System was designed to serve an area of approximately 109 sq miles, but ultimately 218.19 sq miles will be tributary to this system. Except for the Charles River Valley Sewer (which receives the sewage from certain combined sewers), the South System is intended to receive only sanitary sewage. It was designed to take care of the sewage of a population of approximately one million persons on a basis of 300 gal per person per day in 1940. Records of the flow in this system, recorded at the screen house at Nut Island, show that the flow in 1935 averaged about 86.5 mgd.

Table 1(*c*) shows the flow in the South Metropolitan Sewerage System. The average daily flow of sewage from June to November, inclusive, was considerably less than that during the entire year, a condition contrary to that in the North Metropolitan Sewerage System and the Boston Drainage System.

### Quality of Sewage Reaching Nut Island Screen House

Analyses of sewage collected by the Department of Public Health during 1913, 1929, and 1936 at the Nut Island screen house are as reported in Table 2(*c*). The chloride content of the sewage from the South District seldom exceeds 250 ppm indicating that only a small quantity of salt water leaks into the system of this district.

### Effect of the Discharge of Sewage from Boston Main Drainage District and the North and South Metropolitan Sewerage Districts

For a number of years the discharge of sewage at Moon Island from the Boston Main Drainage District has caused objectionable conditions in the vicinity of the outfall because the discharged sewage enters the harbor at considerable velocity on the outgoing tide, at approximately the surface of the water, and spreads over a very large area, causing extensive sleek areas.

The sewage discharged from the North Metropolitan Sewerage District near Deer Island has never caused objectionable conditions except from odor in the vicinity of the Deer Island Light and from floating matter reaching nearby shores. Extensive sleek areas have seldom been noticed in connection with the discharge of sewage from this district. The limited size of the sleek areas is probably due to the rough water in that vicinity.

Although the sewage discharged from the South Metropolitan District does not result in as objectionable conditions as that from the Boston Main Drainage District, it does cause large sleek areas at times, especially in the vicinity of Nut Island on the incoming tide.

In order to ascertain the general condition of the harbor water resulting from the discharge of sewage from these three main outlets, an extensive program of sampling was conducted during the investigation by the Special Commission in 1935 and 1936 and, as far as practicable, these sampling stations conformed with those of previous investigations. Samples of the water were

Fig. 3.—Boston Harbor, Facing Moon Head Outlet

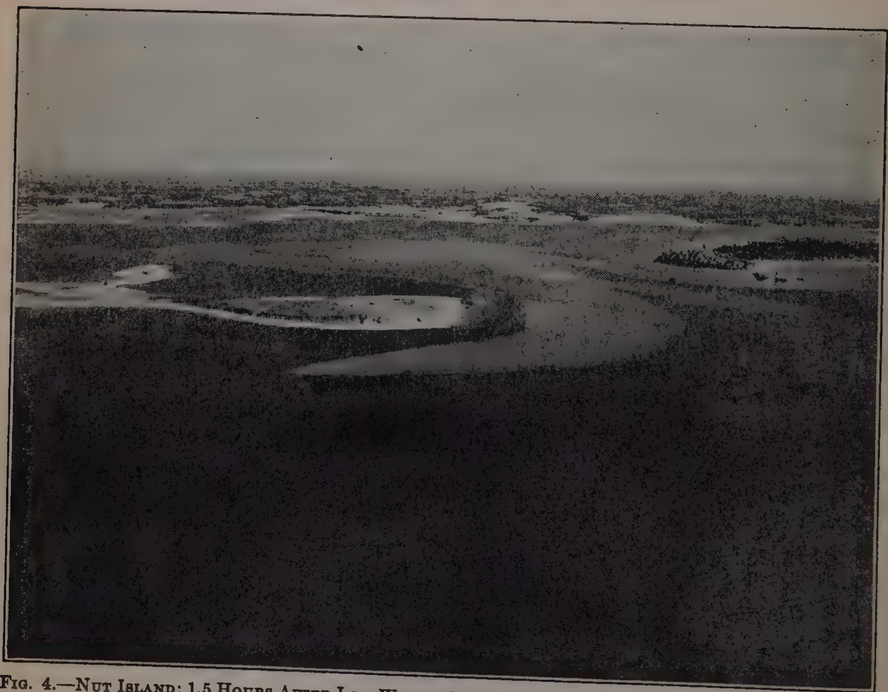

Fig. 4.—Nut Island; 1.5 Hours After Low Water; Northwest Wind; Velocity, 16 Miles per Hour

collected for chemical analysis and bacterial examination, and field tests were made to determine the quantity of dissolved oxygen in the water.

In order to interpret more intelligently the results of the chemical analyses and bacterial examinations, and in order to determine the extent to which sewage matters reach the shores of Boston Harbor and vicinity, extensive float tests were made to compare with the result of previous tests. To determine the extent to which floating matter, especially grease, might travel from these outlets, observations were made of sleek areas from boats, and a number of aerial photographs were taken (see Figs. 3 and 4).

Surface and depth samples were collected along the courses of floats for chemical analysis and bacterial examination; and samples were collected at regular intervals along the shores, especially in the vicinity of bathing beaches.

## Tidal Currents in the Vicinity of the Outfall Sewers

The velocity and direction of the tidal currents were determined by observing the courses taken by floats released at the various outfalls and at other points throughout the harbor. Floats were also used to determine locations better adapted for points of discharge of the sewage. Three types of floats were used for this work—floats of deep submergence, surface floats of shallow submergence, and chips. The depth floats were constructed of 2 in. by 4 in. spruce timbers, 6 ft long, weighted at the bottom to hold the float in a vertical position and to provide submergence, except for 2 in. which extended above the surface of the ground, and which was surmounted with a small signal flag. Each of the floats was equipped with vanes nailed to the sides to prevent whirling. The surface floats were similarly constructed of timber but were only 3 ft to 4 ft long. The chips consisted of small pieces of lath appropriately painted and marked by branding iron with the name and address of the Department of Public Health with the request that they be returned to the Department.

In general, the float tests confirmed the results of previous tests and showed that: (a) The velocity of the tidal flow in Boston Harbor has not changed considerably over a number of years, regardless of certain changes in and about the harbor due to the dredging operations and the filling of large areas for the construction of the Boston Airport, a bulkhead to Castle Island, and the filling in of Shirley Gut; (b) regardless of where sewage is discharged in Boston Harbor it may be expected that floating matter in the sewage will be washed on to the shores in the harbor or on adjacent shores. The courses taken by some of the floats released in various parts of the harbor are shown in Fig. 5, and Figs. 6 and 7 indicate the places where some of the chips were retrieved.

## Results of Chemical Analyses

During the investigations of 1929 and 1930 the Department of Public Health made numerous tests on both the incoming and outgoing tides to determine the dissolved oxygen content in various parts of Boston Harbor, and during the joint investigation of 1935–1936 analyses and tests were made on samples from these same stations and at many additional ones.[3] The dissolved oxygen present in the water during the later investigation was found to

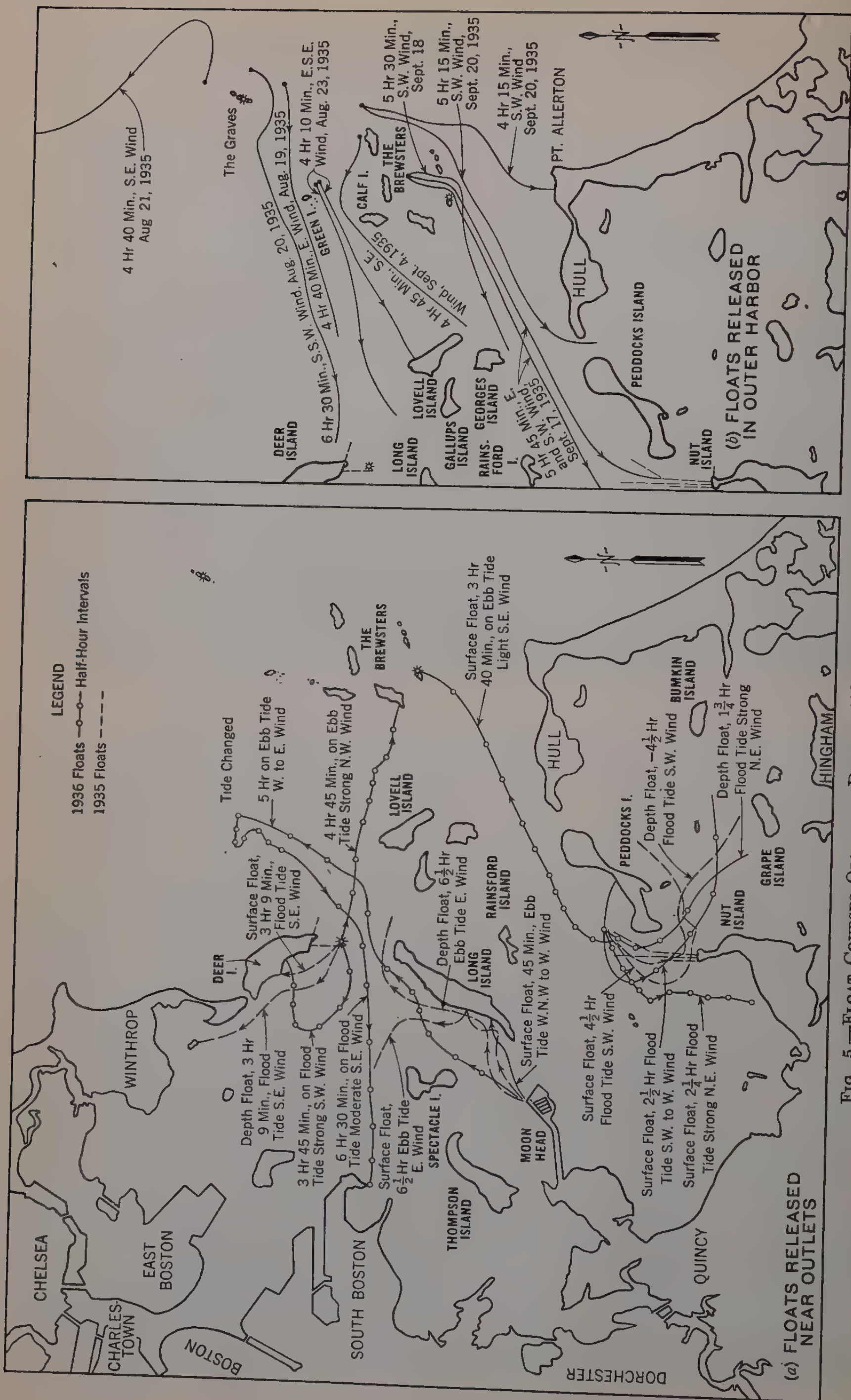


FIG. 5.—FLOAT COURSES OBSERVED DURING 1935–1936 INVESTIGATION IN BOSTON HARBOR

be in excess of that found in 1929 at all stations, and on the incoming tide it was in excess of that found during the 1930 investigation. At no time during the 1935–1936 investigation was the harbor water found to be less than 50%

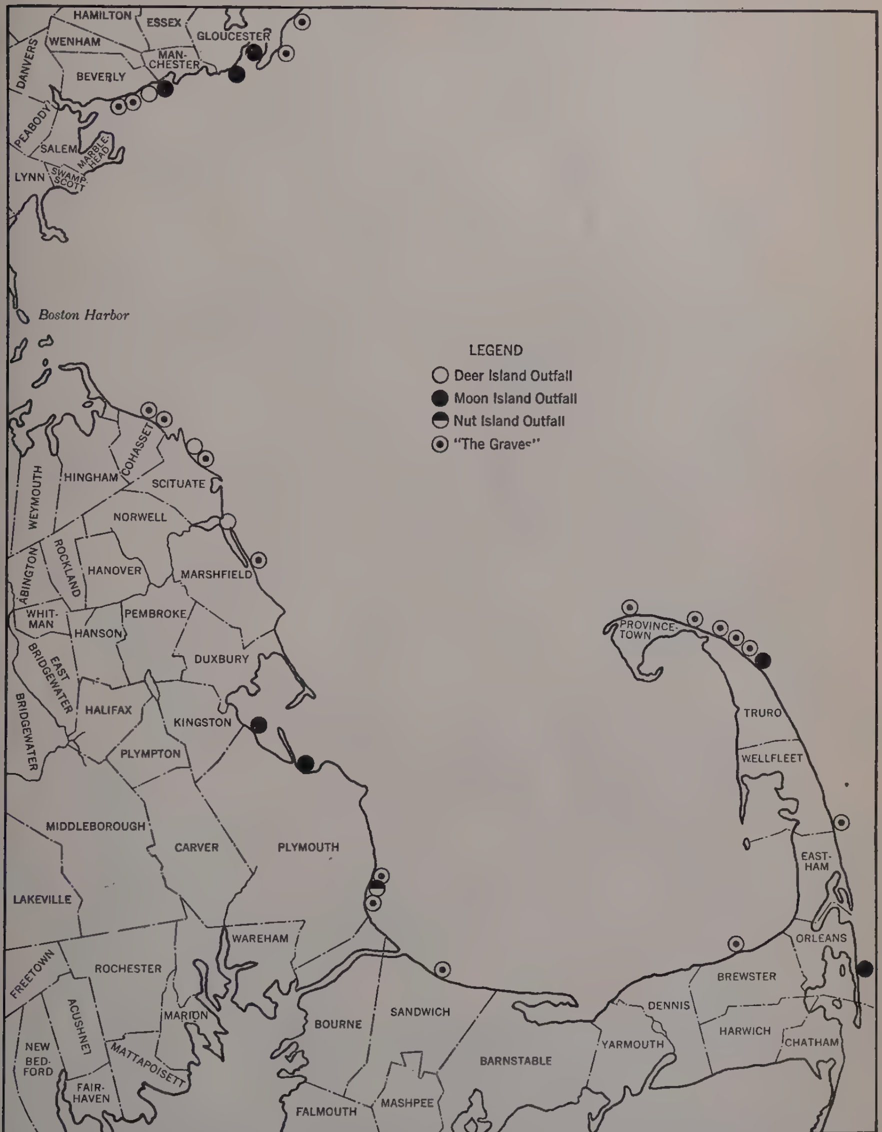


Fig. 6.—Distant Points Where Chips Released at Outfalls, in October, 1935, Were Found

saturated, even in the vicinity of the main sewer outlets, and in the Inner Harbor opposite Fort Point Channel the dissolved oxygen content of the water averaged about 75% saturated. Except for the Inner Harbor the lowest

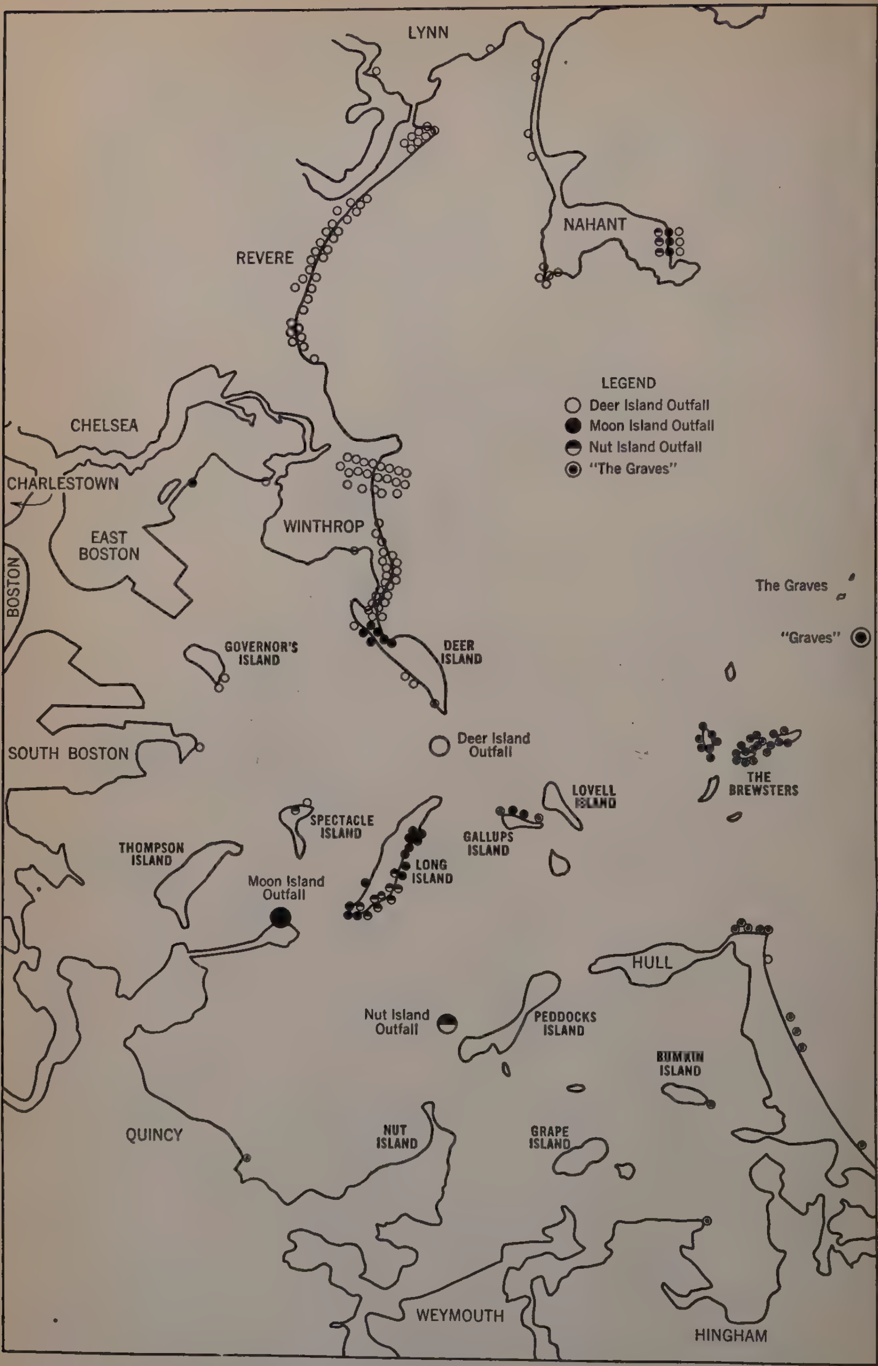


FIG. 7.—NEAR-BY POINTS WHERE CHIPS RELEASED AT OUTFALLS, IN OCTOBER, 1935, WERE FOUND

average dissolved oxygen saturation was 78.3%, which was found at a point a short distance northwest of the pier on Long Island. Determinations also were made[3] of the bio-chemical oxygen demand at various sampling points in the harbor (see Table 4). On the incoming tide in 1935 the oxygen demand was low in all sections of the harbor, the highest being 1.3 ppm in the Outer Harbor and Quincy Bay and the lowest being 0.3 ppm in Dorchester Bay. On the outgoing tide in 1936, the oxygen demand was somewhat higher, with a maximum of 4.1 ppm in Dorchester Bay. At no station where samples were collected did the results show that the dissolved oxygen in the water had been depleted sufficiently to cause a nuisance.

TABLE 4.—Bio-Chemical Oxygen Demand, Five Days at 20° Centigrade (Parts per Million)

| Tide | Year | Inner Harbor | Outer Harbor | Dorchester Bay | Winthrop Bay | Quincy Bay | Hingham Bay | Harbor entrances |
|---|---|---|---|---|---|---|---|---|
| Surface flood | 1935 | 0.8 | 1.3 | 0.3 | 1.2 | 1.3 | 1.0 | 1.0 |
| Surface ebb | 1936 | 2.0 | 0.9 | 4.1 | 1.0 | .... | .... | .... |

For purposes of comparison, results of the dissolved oxygen tests of the waters of New York Harbor and estuaries, and certain estuaries of Boston Harbor, and the North River in Salem are shown in Table 5.

TABLE 5.—Dissolved Oxygen; Various Harbors and Tributaries

| Place | Station | Number of samples | Percentage Saturation | | |
|---|---|---|---|---|---|
| | | | Minimum | Average | |
| | | | | Ebb | Flood |
| New York, N. Y. (1933): | | | | | |
| Hudson River | 42nd Street* | 20 | 20 | 48 | 36 |
| Harlem River | 155th Street | 24 | 0 | 12 | 26 |
| Lower East River | 42nd Street | 32 | 0 | 16 | 21 |
| Upper Bay | Bell Buoy No. 2G | 30 | 21 | 38 | 35 |
| Narrows | ................ | 26 | 29 | 49 | 51 |
| Salem, Mass. (1936): | | | | | |
| North River | Mouth | 6 | 0 | 0.7 | .... |
| Boston, Mass.: | | | | | |
| Chelsea River (1935) | Lower Bridge | 1 | .... | 63.3 | .... |
| Fort Point Channel (1936) | ................ | 6 | 55.9 | 63.7 | 87.5 |
| Charles River (1935) | ................ | 1 | .... | 62.2 | .... |

* North River.

Although tests for B.O.D. have been made by the Department of Public Health of Massachusetts only during recent years, analyses for free and albuminoid ammonia have been made for many years, in some instances dating back to the early nineties in connection with pollution of tidal waters. Accordingly, it has been the practice of the Department to continue these analyses in order that comparisons may be made between these results and the old results where practicable. They show that, in most cases, the organic matter in the harbor waters during the 1935–1936 investigation[3] was about the same as in the 1929–1930 investigation. A slightly greater degree of contamination

was found in 1936 at the entrance of Hingham Bay near Windmill Point, at the entrance of the Harbor north of Point Allerton, in the North Channel northeast of Deer Island, and in Winthrop Bay near Shirley Gut. The condition of the water in the Inner Harbor, in the vicinity of Boston proper, was somewhat better in 1935–1936—especially on the outgoing tide—than in 1929–1930. On the incoming tide in 1935 and 1936, the content of organic matter was greater, in Dorchester Bay, than in 1929 and 1930; and, on the outgoing tide, it was less. Because of the large concentration of dissolved oxygen in the harbor waters, the ammonia results were of greater assistance in determining the extent of pollution throughout the harbor than were the results of the tests of dissolved oxygen. In Fig. 8 the 1935–1936 results are shown in diagrammatic form as a means of comparison with the dissolved oxygen determinations.

## Bacterial Examinations

Earlier investigations as well as those of 1935–1936[5] show that the chief nuisances resulting from the disposal of sewage in Boston Harbor are those created by bacterial pollution and by floating matter from the sewage reaching the shores, especially the shores used for bathing.

The 1935 bacterial examinations showed that on the incoming tide the water at the surface of the Outer Harbor in the vicinity of Deer Island and Nut Island outlets contained large numbers of bacteria characteristic of sewage, and that the contaminated areas extended from the harbor entrance to the westerly part of Hingham Bay and into the easterly section of Winthrop Bay in the vicinity of Deer Island and Point Shirley. They also showed the presence of a large number of such bacteria in that section of the Charles River between the Inner Harbor and the Charles River Dam and, on the incoming tide, in the waters of the Outer Harbor entrances in the vicinity of Lovell, Gallups, and Georges islands; and, in a small area east of Deer Island. The samples collected on the outgoing tide contained large numbers of bacteria characteristic of sewage between the Inner Harbor and the Charles River Dam, in Fort Point Channel and along the Boston water-front. The waters of Dorchester Bay, in the vicinity of the Naval Air Base at Squantum, and in the vicinity of Commercial Point, contained large numbers of bacteria characteristic of pollution, a condition probably due to local contamination. Quincy Bay in the vicinity of Moon Island contained large numbers of bacteria on the outgoing tide, a condition due to the discharge of sewage from the outfall works of the Boston Main Drainage system. In the vicinity of Deer Island, water on the outgoing tide contained large numbers of bacteria in the northeasterly section of the Outer Harbor and at the entrances to the harbor. The results of samples collected in 1935 and 1936 show that on the outgoing and incoming tides the water, at a 20-ft depth in the vicinity of the outlets, contained a larger number of bacteria than at the surface; but at points close to the shore the number present was about the same in the surface and depth samples. On the incoming tide large areas of the Outer Harbor, Winthrop Bay, and parts of the Inner Harbor are contaminated by the discharge of sewage from Deer Island, and

[5] "Report of the Special Commission on the Investigation of the Discharge of Sewage into Boston Harbor and Its Tributaries," House Doc. No. 1600, December, 1936, p. 221.

the entrance to the harbor is polluted, in addition, by the discharge of sewage from the Moon Island outlet. The 1936 samples showed that, on the incoming tide, the waters in the vicinity of the Deer Island and Nut Island sewer outlets, and in the vicinity of Long and Rainsford islands, are grossly polluted. Pollution in the vicinity of Long and Rainsford islands may be caused partly by the discharge of sewage from Long Island and from the return of sewage which had previously been discharged from Moon Island on the outgoing tide. This area appeared to be highly polluted on both the outgoing and incoming tides. The depth samples collected on the incoming tide showed a polluted area off the

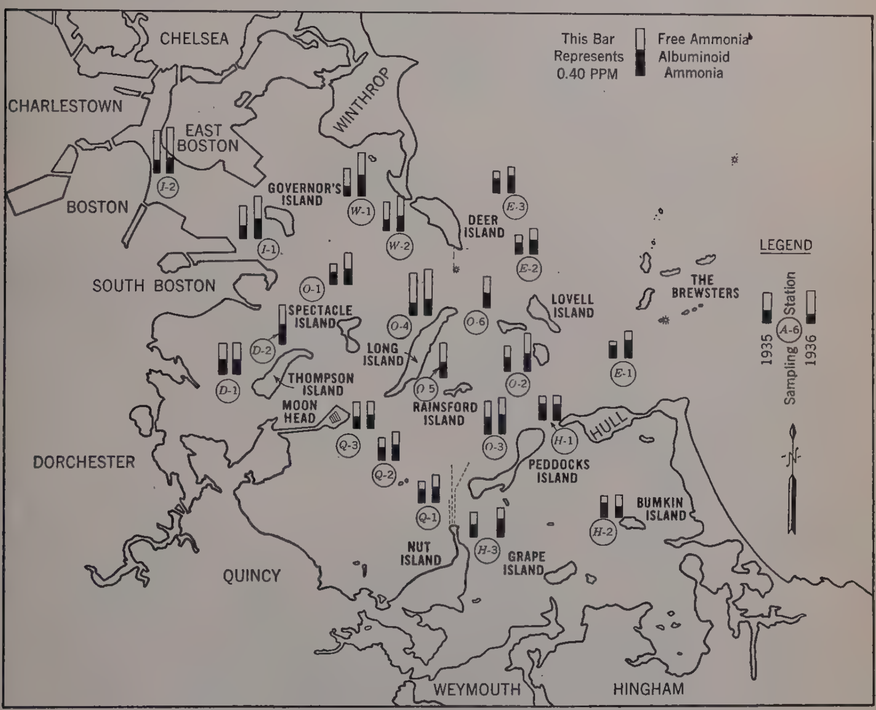


FIG. 8.—AVERAGE CONTENT OF ALBUMINOID AND AMMONIA DURING FLOOD AND EBB TIDES

northwesterly shore of Thompson Island and between Gallups and Long islands and that under certain conditions the sewage from the Nut Island outfall sewer approaches the shore along Houghs Neck in Quincy. In general, on the outgoing tides the polluted areas were greater in 1936 than in 1935; but this condition may be explained in part by the fact that samples from a number of new sampling points between Long and Rainsford islands, between Long and Gallups islands, and north of Thompson Island, collected during the 1936 investigation, were not collected during the 1935 investigation.

Fig. 9 shows the percentage of the 0.1-cu cm portions of the samples collected during the investigation of 1936 containing bacteria of the *Coli-Aerogenes*

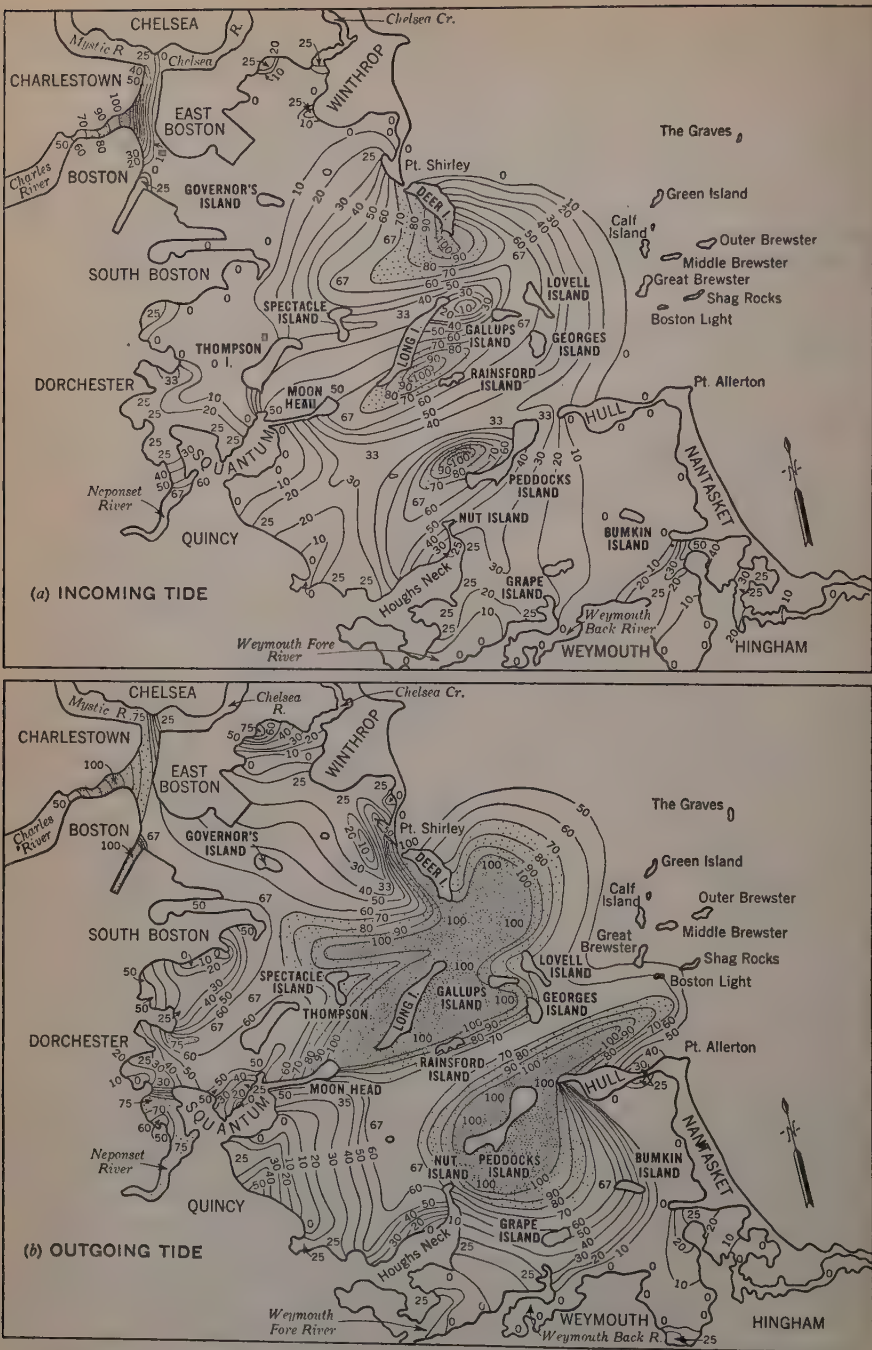


FIG. 9.—DETERMINATIONS OF MEMBERS OF THE *COLI-AEROGENES* GROUP FROM SURFACE SAMPLE

groups. For conveniences, the lines connecting zones of equal *Coli-Aerogenes* density have been termed "isocols."

### Bacterial Condition of the Harbor Bottom

In connection with the bacterial investigation of the waters of Boston Harbor, samples were collected of the mud and silt along the bottom of the harbor at the regular sampling stations. In practically all cases the samples of mud contained many more bacteria, characteristic of pollution, than the waters at the surface, indicating that bacteria are deposited in the mud with sewage solids and are not washed out to sea with the outgoing tide. These results verify previous ones obtained by the Department of Public Health in the examination of certain bathing beaches where it was found that, although the waters over the beaches may sometimes be free from organisms of the *Coli-Aerogenes* group, the silt on the bottom at these beaches may contain large numbers of these organisms.

### Recreational and Other Uses of Boston Harbor

Boston Harbor is used extensively for recreation and, to a limited degree, for the taking of shellfish for food. Although the harbor waters were not found to be so grossly polluted that nuisance from odor resulted, it is important to consider the effect of this pollution on waters used for bathing and from which shellfish are taken for food. Certain consideration should be given to the use of these waters for yachting. During the investigation by the Special Commission in 1935–1936, it was found that there were 21 yacht clubs along the shores from Point Shirley in Winthrop to Windmill Point in Hull, having a total membership of approximately 3 200 persons, and some 1 370 pleasure boats. It is interesting to note that at 6 of these clubs sewage was discharged directly into the harbor.

The sanitary surveys of 1935–1936 also showed that there were 29 areas between Point Shirley in Winthrop and Windmill Point in Hull where bathing beaches had been established. Of this number only 12 were under the control of the Metropolitan District Commission or the City of Boston, and at many of the areas no bath-houses or close supervision were provided. It was estimated that approximately 100 000 persons frequent the popular beaches on some pleasant Sundays and that 15 000 additional persons use those beaches where no record of attendance is kept.

In order to compare the results of the bacterial examination of the samples of the water collected at the beaches of Boston Harbor and some of those outside the harbor, Table 6 is presented. It is apparent that the beaches in Boston Harbor are more highly polluted than those outside but not as grossly polluted as at some beaches on streams tributary to the harbor. Some of the areas in Boston Harbor, now used for bathing, must be abandoned for that purpose, or a considerable sum of money must be expended to treat the sewage, or to remove it by extending the present sewer outlets to points more remote from the beaches. The investigation failed to disclose any illness due to bathing in these waters; but, regardless of the lack of such evidence, it is obvious that

there must be always a danger to public health where bathing is permitted in the vicinity of sewer outlets. Accordingly, the Special Commission stated in its report to the Legislature[6] that in view of the numerous clean beaches

TABLE 6.—AVERAGE BACTERIAL ANALYSIS OF PUBLIC BEACHES IN BOSTON HARBOR COMPARED WITH THOSE OUTSIDE THE HARBOR

| Location* | Bacteria per Cubic Centimeter | | | *Coli-Aerogenes* | | | | | |
|---|---|---|---|---|---|---|---|---|---|
| | Four days 20° Centigrade | 24 Hours 37° Centigrade | | Percentage Positive in: | | | | | |
| | | Total | Reduction | 0.001 cubic centimeters | 0.01 cubic centimeters | 0.1 cubic centimeters | 1.0 cubic centimeter | 10 Cubic Centimeters Positive | 10 Cubic Centimeters Negative |
| 1936: | | | | | | | | | |
| Wood Island Park, public, Boston | 421 | 20 | 4 | 0 | 0 | 0 | 37.5 | 87.5 | 12.5 |
| Marine Park, public, Boston | 741 | 31 | 8 | 0 | 0 | 25 | 62.5 | 100 | 0 |
| Carson Beach, public, Boston | 1 004 | 137 | 84 | 12.5† | 12.5 | 37.5 | 67.5 | 87.5 | 12.5 |
| Savin Hill, public | 2 501 | 31 | 16 | 0 | 0 | 25 | 87.5 | 100 | 0 |
| Tenean Beach, public, Boston | 780 | 49 | 21 | 0 | 0 | 37.5 | 75 | 100 | 0 |
| Wollaston Beach, North Beach (Atlantic Street) | 1 210 | 105 | 57 | 0 | 12.5 | 12.5 | 37.5 | 100 | 0 |
| Wollaston Center Beach (Wollaston Yacht Club) | 1 966 | 407 | 300 | 12.5† | 25 | 37.5 | 50 | 100 | 0 |
| Wollaston South Beach (Fenno Street) | 911 | 16 | 3 | 0 | 0 | 0 | 12.5 | 87.5 | 12.5 |
| Merrymount Beach, private | 1 411 | 218 | 125 | 0 | 0 | 25 | 37.5 | 75 | 25 |
| Weymouth Municipal, public | 596 | 56 | 10 | 0 | 0 | 0 | 50 | 87.5 | 12.5 |
| Hingham Beach, public (outside beaches) | 1 059 | 22 | 4 | 0 | 0 | 12.5 | 25 | 87.5 | 12.5 |
| Salem, salt water pool (6) | 2 030 | 5 | 0.3 | 0 | 0 | 0 | 0 | 33.3 | 66.7 |
| Winthrop, outside beach | 1 186 | 11 | 3 | 0 | 0 | 0 | 12.5 | 87.5 | 12.5 |
| Swansea, Long Beach (5) | 884 | 5 | 0 | 0 | 0 | 0 | 0 | 100 | 0 |
| Marblehead, Preston Beach (3) | 2 507 | 5 | 0.3 | 0 | 0 | 0 | 0 | 33.3 | 66.7 |
| 1933: | | | | | | | | | |
| Revere Beach, South Beach (14) | 2 063 | 32 | 4 | 0 | 0 | 14.3 | 28.6 | 78.5 | 21.5 |
| Revere Beach, North Beach (13) | 6 212 | 101 | 7 | 0 | 0 | 7.7 | 23.1 | 84.6 | 15.4 |
| Revere Beach, Center Beach (14) | 1 662 | 227 | 4 | 0 | 0 | 7.1 | 28.6 | 78.5 | 21.5 |
| 1931: | | | | | | | | | |
| Barnstable-Hyannis (4) | 5 730 | 279 | 35 | 0 | 0 | 0 | 0 | 100 | 0 |
| Fairhaven (6) | 2 147 | 15 | 3 | 0 | 0 | 0 | 0 | 83.7 | 16.3 |
| 1928: | | | | | | | | | |
| Nantasket Beach (3) | 1 260 | 43 | 8 | 0 | 0 | 0 | 33 | 100 | 0 |

* Eight samples, except as shown in parentheses. † 0 in 0.0001 cubic centimeters.

available to the public there was no good reason to permit bathing near sewer outlets, and that public bathing along the water-front of Boston Harbor and its estuaries and tributaries should be restricted to such areas as may meet with the approval of the Board of Health of the city or town in which such bathing site is situated, or with the approval of the State Department of Public Health.

[6] "Report of the Special Commission on the Investigation of the Discharge of Sewage into Boston Harbor and Its Tributaries," House Doc. No. 1600, December, 1936, p. 318.

## Taking of Shellfish from Boston Harbor

The taking of shellfish from Boston Harbor is regulated by acts of the Massachusetts Legislature. Also in accordance with Massachusetts laws, the Department of Public Health has permitted master diggers, under permits issued by the Supervisor of Marine Fisheries, to take shellfish from certain areas in Boston Harbor for treatment purposes at an approved shellfish puri-

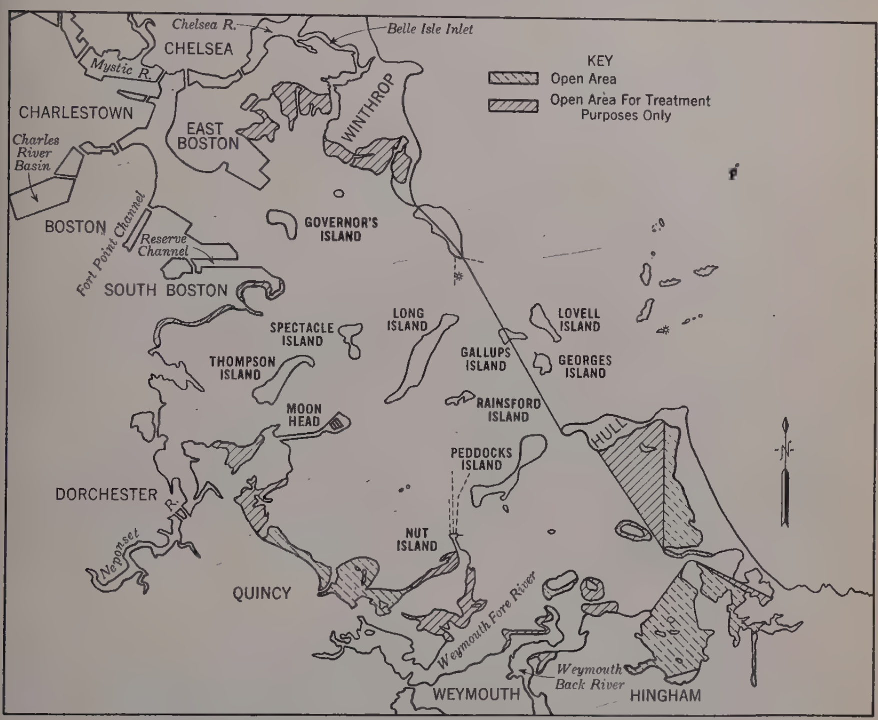


Fig. 10.—Shellfish Areas in Boston Harbor as of July 1, 1937

fication plant. Fig. 10 shows the areas in Boston Harbor where shellfishing was permissible as of July 1, 1937, either for treatment purposes or for use as food without treatment.

## Experimental Sewage Treatment Works

The ordinary routine investigation to determine the most appropriate method of sewage treatment would consider the capacity of the water in which the effluent is to be discharged to replenish the oxygen that would be consumed by the bio-chemical processes of the purification of the sewage. There is no question of the capability of the tidal prism of Boston Harbor on the outgoing and incoming tide, and the waters of the streams tributary to this harbor, to supply sufficient oxygen to replenish that consumed by the disposal of sewage in the harbor.

Bacterial examinations, tests for dissolved oxygen, and chemical analyses, were made of the waters of Boston Harbor, and of sewage discharged from the North and South Metropolitan Sewerage districts and the Boston Main Drainage District. They indicated that objectionable conditions can be removed if works are provided for the removal of certain suspended matters and grease from the sewage before it is discharged. The settled sewage may be chlorinated should further reduction of bacteria of the *Coli-Aerogenes* group become necessary.

To determine the most practicable method of treating this sewage, experimental plants were constructed at Deer Island for use in connection with the sewage of the North Metropolitan Sewerage District, at Nut Island in connection with the sewage of the South Metropolitan District, and at Calf Pasture and Moon Island in connection with the sewage from the Boston Main Drainage District. The small-scale experimental plants consisted of fine screens, a 1 000-gal settling tank, and a 650-gal tank used for pre-aeration of the sewage to aid in the removal of grease. The settling tank was provided with suitable baffles for removing the floating solids. The detention period varied from 1 to 6 hr. In addition, experiments were made to determine the reduction of the number of bacteria of the *Coli-Aerogenes* group by discharging the sewage into salt water. The pump used had a rated capacity of 40 gal per min but under the conditions of operation did not exceed 30 gal per min.

The experimental apparatus used in connection with the South Metropolitan District was similar to that of the North, as was also the apparatus used in connection with the experiments on the sewage from the Boston Main Drainage District at Calf Pasture. Provisions were also made for large-scale experiments on Moon Island whereby one of the large storage tanks having a capacity of about 8 million gal was utilized.

## Experiments at Calf Pasture

The results of the experiments on plain sedimentation, including 1, 1.5, and 2 hr of detention on a continuous flow basis and 3, 4, 5, and 6 hr of detention on an intermittent basis are shown in Table 7(*a*).

As was expected the suspended solid removal in these experiments was about 30% in 1 hr and 50% in 2 hr. The additional removal after detention periods greater than 2 hr was small, being only a total of 62.5% in 6 hr. Removal of the organic matter as represented by the bio-chemical oxygen demand (about 19% with a 2-hr detention period) was relatively low. The removal of the nitrogenous matter as represented by the albuminoid ammonia and Kjeldahl nitrogen determinations was 21% and 21.9%, respectively, for a 2-hr detention period. In these experiments there was little relation between the grease removal and the detention period but the results indicate that about 40% of the grease can be removed by skimming during a plain sedimentation period of 2 hr.

The experiments made to determine the quantity of grease that could be skimmed from the surface of the tanks with aeration prior to sedimentation showed that a slightly greater proportion of grease could be removed with

pre-aeration. The additional detention period resulting from aeration was about one-third of the total aeration period. Pre-aeration, however, resulted in a greater reduction in the percentage of suspended solids and nitrogenous matter. The percentage of removal of the suspended solids by plain sedimentation was about 50% for a 2-hr detention period, whereas with pre-aeration, the removal was about 57% for 2.25-hr periods. The results of the sedimentation tests, with pre-aeration, are shown in Table 7(*b*).

The sewage used in the experiments for the Boston Main Drainage System was obtained at the Calf Pasture Pumping Station from a chamber at the entrance to the deposit sewer. The experiments were made on the sewage from this district 24 hr per day for a 7-day week from September 26 to November 21, 1935.

### Experiments on the Sewage of the North Metropolitan System

The Deer Island experimental plant was operated for the most part 16 hr a day, from 7 A.M. to 11 P.M. Early in the morning the sewage was generally at such a low level that it was impracticable to raise it with the equipment used. The period of experimentation was from September 30 to November 9, 1935, and from May 15 to July 31, 1936.

In the plain settling tests the average reduction in suspended solids was about 45% in 1.5 hr, 51% in 2 hr, and 57% in 3 hr. The average B.O.D. reduction was about 25% during these periods and the reduction of nitrogenous matter, as represented by the Kjeldahl nitrogen determination, was 33 per cent. Approximately 40% of the grease in the sewage was removed in a detention period of 2 hr. Table 7(*d*) shows the results of the plain sedimentation experiments.

### Experiments on Sewage of the South Metropolitan Sewerage District

The experiments on the sewage from the South Metropolitan Sewerage District were extended over a period from 7 A.M. to 11 P.M. from Monday to Friday, inclusive, from August 25 to September 19 and October 2 to October 9, 1936. Because of the short period of operation a few of the samples were 4-hr composites instead of the usual 8-hr composites. The sewage for these experiments was taken from the main outfall sewer through a manhole to the grit-chamber at the entrance to the screen house at Nut Island. The efficiency of settling was much greater in connection with the sewage of the South Metropolitan System than from the Boston Main Drainage System although the analyses showed that the sewage of these two systems was similar. The reduction in suspended solids during a 2-hr detention period was 69% as compared with 50% ordinarily obtained. The reduction in the oxygen demand, organic nitrogen, and oxygen consumed, was 40%, 40%, and 36%, respectively. It is probable that if the experiments had been extended over a sufficient period, lower reductions, more consistent with the results of the experiments on the sewage of the Boston Main Drainage System, would have been obtained. Table 7(*e*) shows the results of plain sedimentation experiments on the sewage of the South Metropolitan Sewerage System.

## TABLE 7.—Observations of Plain Sedimentation

| Detention period, in hours | Number of samples* | Suspended Solids: Parts per Million: Raw sewage | Suspended Solids: Parts per Million: Effluent | Suspended Solids: Per centage reduction | Bio-chemical Oxygen Demand: Number of samples* | Bio-chemical Oxygen Demand: Parts per Million: Raw sewage | Bio-chemical Oxygen Demand: Parts per Million: Effluent | Bio-chemical Oxygen Demand: Per centage reduction | Kjeldahl Nitrogen: Parts per Million: Raw sewage | Kjeldahl Nitrogen: Parts per Million: Effluent | Kjeldahl Nitrogen: Per-centage reduction | Total Albuminoid Ammonia: Parts per Million: Raw sewage | Total Albuminoid Ammonia: Parts per Million: Effluent | Total Albuminoid Ammonia: Per-centage reduction | Oxygen Consumed: Parts per Million: Raw sewage | Oxygen Consumed: Parts per Million: Effluent | Oxygen Consumed: Per-centage reduction | Fats: Number of samples* | Fats: Parts per Million: Raw sewage | Fats: Parts per Million: Effluent | Fats: Per-centage reduction |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| (1) | (2) | (3) | (4) | (5) | (6) | (7) | (8) | (9) | (10) | (11) | (12) | (13) | (14) | (15) | (16) | (17) | (18) | (19) | (20) | (21) | (22) |
| | | | | | | | | | (*a*) Calf Pasture; without Pre-Aeration | | | | | | | | | | | | |
| 1 | 19 | 153 | 108 | 29.4 | 17 | 173 | 143 | 17.3 | 8.9 | 7.4 | 16.9 | 4.21 | 3.40 | 19.3 | 56 | 48 | 14.3 | 18 | 54 | 37 | 31.5 |
| 1.5 | 9 | 159 | 92 | 42.2 | .... | 173 | 143 | 17.3 | 8.7 | 7.7 | 11.5 | 4.41 | 3.68 | 16.5 | 61 | 46 | 24.6 | 8 | 39 | 22 | 43.6 |
| 2 | 9 | 190 | 93 | 50.9 | 7 | 161 | 130 | 19.3 | 8.7 | 6.8 | 21.9 | 4.23 | 3.34 | 21.0 | 56 | 42 | 25.0 | .... | 46 | 28 | 39.1 |
| 3 | 18 | 156 | 77 | 50.6 | 17 | 192 | 142 | 26.1 | 9.6 | 6.6 | 31.3 | 4.40 | 3.14 | 28.6 | 58 | 47 | 19.0 | 17 | 52 | 29 | 44.2 |
| 4 | 12 | 136 | 70 | 48.5 | 10 | 186 | 136 | 26.9 | 8.7 | 6.2 | 28.8 | 4.28 | 2.78 | 35.0 | 53 | 46 | 13.2 | .... | 45 | 28 | 37.8 |
| 5 | 11 | 172 | 82 | 52.3 | .... | 203 | 139 | 31.5 | 9.3 | 6.9 | 25.8 | 4.47 | 3.18 | 26.6 | 59 | 48 | 18.7 | .... | 57 | 34 | 40.3 |
| 6 | 9 | 173 | 65 | 62.5 | .... | 174 | 115 | 33.9 | 9.0 | 5.3 | 41.2 | 4.27 | 2.66 | 37.7 | 56 | 42 | 25.0 | .... | 62 | 26 | 58.0 |
| | | | | | | | | | (*b*) Calf Pasture; with Pre-Aeration | | | | | | | | | | | | |
| 0.75 | 6 | 132 | 85 | 35.6 | .... | 163 | 146 | 10.4 | 8.7 | 8.1 | 6.9 | 3.60 | 3.32 | 7.8 | 52 | 44 | 15.4 | .... | 41 | 31 | 24.4 |
| 1.5 | 4 | 169 | 88 | 48.0 | .... | 203 | 160 | 21.2 | 13.1 | 9.0 | 31.3 | 4.85 | 3.42 | 29.5 | 56 | 48 | 14.3 | .... | 61 | 41 | 32.8 |
| 2.25 | 8 | 169 | 72 | 57.3 | .... | 165 | 121 | 26.7 | 11.9 | 6.9 | 42.0 | 5.43 | 3.42 | 37.0 | 56 | 44 | 21.4 | .... | 58 | 31 | 46.6 |
| 3 | 6 | 168 | 75 | 55.3 | .... | 174 | 114 | 35.5 | 10.7 | 6.9 | 35.5 | 4.83 | 2.95 | 38.9 | 56 | 43 | 23.2 | .... | 50 | 36 | 28.0 |
| | | | | | | | | | (*c*) Deer Island (1935); with Pre-Aeration | | | | | | | | | | | | |
| 1 | 3 | 224 | 164 | 26.8 | .... | 269 | 244 | 9.3 | 12.9 | 10.5 | 18.9 | 7.30 | 6.43 | 11.9 | 80 | 74 | 7.5 | .... | 67 | 56 | 16.4 |
| 1.5 | 9 | 238 | 126 | 47.1 | .... | 260 | 190 | 26.9 | 17.4 | 13.6 | 35.0 | 8.03 | 5.70 | 29.0 | 79 | 64 | 19.0 | .... | 75 | 38 | 49.3 |
| 2.25 | 3 | 233 | 123 | 47.2 | .... | 234 | 171 | 31.2 | 17.5 | 10.3 | 41.2 | 7.17 | 4.93 | 31.2 | 81 | 71 | 12.4 | .... | 41 | 33 | 19.5 |
| 3 | 5 | 238 | 106 | 55.5 | .... | 250 | 168 | 32.8 | 16.5 | 11.1 | 32.7 | 7.56 | 4.84 | 36.0 | 80 | 66 | 17.5 | .... | 66 | 31 | 53.0 |

TABLE 7.—(*Continued*)

| Detention period, in hours | Number of samples* | Suspended Solids | | | Bio-chemical Oxygen Demand | | | | Kjeldahl Nitrogen | | | Total Albuminoid Ammonia | | | Oxygen Consumed | | | Fats | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | | Parts per Million | | Percentage reduction | Number of samples* | Parts per Million | | Percentage reduction | Parts per Million | | Percentage reduction | Parts per Million | | Percentage reduction | Parts per Million | | Percentage reduction | Number of samples* | Parts per Million | | Percentage reduction |
| | | Raw sewage | Effluent | | | Raw sewage | Effluent | | Raw sewage | Effluent | | Raw sewage | Effluent | | Raw sewage | Effluent | | | Raw sewage | Effluent | |
| (1) | (2) | (3) | (4) | (5) | (6) | (7) | (8) | (9) | (10) | (11) | (12) | (13) | (14) | (15) | (16) | (17) | (18) | (19) | (20) | (21) | (22) |
| (*d*) Deer Island (1935 and 1936); without Pre-Aeration | | | | | | | | | | | | | | | | | | | | | |
| 1 | 14 | 170 | 97 | 42.9 | .... | 218 | 181 | 17.0 | 13.0 | 9.8 | 24.6 | .... | .... | .... | 72 | 62 | 13.9 | .... | 49 | 30 | 38.8 |
| 1.5 | 16 | 176 | 97 | 44.9 | .... | 224 | 167 | 25.4 | 12.5 | 8.5 | 32.0 | .... | .... | .... | 73 | 61 | 16.5 | .... | 40 | 28 | 30.0 |
| 2 | 15 | 171 | 83 | 51.4 | .... | 183 | 138 | 24.6 | 11.9 | 8.0 | 32.8 | .... | .... | .... | 71 | 58 | 18.3 | .... | 49 | 28 | 42.9 |
| 3 | 16 | 187 | 80 | 57.2 | .... | 250 | 181 | 27.6 | 13.3 | 8.8 | 33.8 | .... | .... | .... | 73 | 58 | 20.6 | .... | 58 | 36 | 38.0 |
| 4 | 16 | 166 | 66 | 60.3 | .... | 198 | 148 | 25.3 | 12.5 | 8.2 | 34.4 | .... | .... | .... | 75 | 58 | 22.7 | .... | 36 | 17 | 52.8 |
| 5 | 10 | 192 | 58 | 69.9 | .... | 178 | 120 | 32.6 | 11.5 | 7.4 | 25.6 | .... | .... | .... | 66 | 49 | 25.8 | .... | 41 | 21 | 48.8 |
| 6 | 10 | 147 | 55 | 62.6 | .... | 185 | 139 | 24.9 | 12.1 | 8.2 | 32.2 | .... | .... | .... | 76 | 61 | 19.8 | .... | 42 | 21 | 50.0 |
| (*e*) Nut Island (1936); without Pre-Aeration | | | | | | | | | | | | | | | | | | | | | |
| 1 | 10 | 195 | 73 | 63 | .... | 160 | 117 | 27 | 11.4 | 6.9 | 40 | 4.98 | 2.99 | 40 | 54.0 | 38.7 | 28 | .... | 59 | 38 | 36 |
| 1.5 | 6 | 168 | 62 | 63 | .... | 118 | 81 | 31 | 10.6 | 6.9 | 35 | 3.63 | 2.70 | 26 | 54.0 | 34.0 | 37 | .... | 46 | 28 | 39 |
| 2 | 10 | 190 | 59 | 69 | .... | 134 | 80 | 40 | 10.7 | 6.4 | 40 | 4.00 | 1.98 | 51 | 58.8 | 37.6 | 36 | .... | 48 | 24 | 50 |
| 3 | 9 | 150 | 55 | 63 | .... | 126 | 76 | 40 | 9.2 | 5.8 | 37 | 3.64 | 2.28 | 37 | 51.3 | 33.0 | 36 | .... | 57 | 30 | 47 |
| 4 | 7 | 136 | 51 | 63 | .... | 115 | 73 | 36 | 9.1 | 6.0 | 34 | 3.27 | 2.04 | 38 | 47.3 | 32.0 | 32 | .... | 51 | 31 | 39 |
| 5 | 2 | 121 | 28 | 77 | .... | 84 | 42 | 50 | 6.0 | 3.8 | 37 | 2.15 | 1.00 | 54 | 41.0 | 28.0 | 32 | .... | 25 | 15 | 40 |
| 6 | 5 | 145 | 58 | 60 | .... | 114 | 71 | 38 | 8.3 | 5.8 | 30 | 2.74 | 2.18 | 20 | 43.0 | 31.0 | 28 | .... | 51 | 36 | 29 |

* For number of samples in fat determinations see Columns (6) and (19).

## Summary of the Results of Sedimentation Experiments

Pre-aeration of the sewage to aid in grease removal was not successful in connection with sewage of the Boston Main Drainage System and the North Metropolitan Sewerage System. It is probable that the aeration does help to separate the grease from the settleable solids; but it is doubtful if, in these experiments, it was of sufficient assistance in increasing the over-all removal of grease to warrant the additional expense.

These experiments, confirming the result of other similar experiments, show that the greatest reduction of suspended solids occurs during the first hour and that with increasing detention periods the increments of removal become less and less.

In general, the results obtained from all three stations were similar and indicated that about 50% of the suspended solids could be removed in a 2-hr detention period. Calculated from the removal of suspended solids, the daily quantities of sludge that would be removed from the sewage of the three sewerage systems on the basis of 1935 flows is shown in Table 8.

TABLE 8.—Quantity of Sludge Expected per Day

| System | Dry solids, in tons | Sludge (95% moisture), in gallons | Sludge (80% moisture), in cubic feet |
|---|---|---|---|
| Boston Main Drainage | 25 | 114 000 | 3 800 |
| North Metropolitan Sewerage | 33 | 153 000 | 5 120 |
| South Metropolitan Sewerage | 30 | 138 000 | 4 610 |
| Totals | 88 | 405 000 | 13 530 |

## Experiments with Fine Screens

The solids obtained by passing the sewage through fine screens, 18 in. by 21 in., which had openings of $\frac{1}{16}$ in., $\frac{3}{32}$ in., and $\frac{1}{8}$ in., by 2 in., is shown in Table 9. The nature of the screenings is described as follows:

| Experiment station | Screenings |
|---|---|
| Calf Pasture | Garbage and paper, with leaves after a heavy rain |
| Deer Island | Garbage and paper with some hair and bristle |
| Nut Island | Garbage and paper |

Because the Boston Main Drainage System is relatively short, more screenings were obtained from sewage of this system than from that of the larger North and South Metropolitan Systems where the larger particles in the sewage have an opportunity to become well disintegrated before reaching the screen chambers.

The weather during the tests was about normal except during the day of September 18, 1936, when the $\frac{1}{16}$-in. screens were in use at Nut Island. At this time 4.64 in. of rain fell and screenings were obtained at the rate of 143 lb of dry material per million gallons of sewage.

Large particles of floating and suspended solids were removed with the use of fine screens. Only a small quantity of the grease (one of the main offenders

in rendering the harbor waters unsightly) was retained on the screens, and it is apparent that fine screens without additional help are insufficient to improve the objectionable appearance of the harbor satisfactorily. Only about 5% to 8% of suspended solids were removed by the fine screens as compared with about 50% removal by plain sedimentation.

TABLE 9.—EXPERIMENTAL RESULTS, FINE SCREEN TESTS

| Experiment station | Year | (a) 1/16-Inch Screen: Total hours of test | (a) Dry Screenings: Pounds | (a) Dry Screenings: Pounds per million gallons | (a) Percentage reduction in suspended solids | (b) 3/32-Inch Screen: Total hours of test | (b) Dry Screenings: Pounds | (b) Dry Screenings: Pounds per million gallons | (b) Percentage reduction in suspended solids | (c) 1/8-Inch Screen: Total hours of test | (c) Dry Screenings: Pounds | (c) Dry Screenings: Pounds per million gallons | (c) Percentage reduction in suspended solids |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| Calf Pasture | 1935 | 167.5 | 21.23 | 106.5 | 7.5 | 131.5 | 14.46 | 92.0 | 7.1 | 102.5 | 10.24 | 83.3 | 5.7 |
| Deer Island | 1935 | 60.0 | 5.92 | 74.9 | 3.8 | 55.8 | 5.32 | 59.3 | 2.7 | .... | .... | .... | .... |
| Deer Island | 1936 | 60.9 | 9.16 | 108.0 | 7.4 | 77.5 | 5.39 | 52.0 | 3.3 | 80.9 | 4.04 | 32.9 | 3.2 |
| Nut Island | 1936 | 33.5 | 4.70 | 75.0 | 5.5 | 27.5 | 2.40 | 52.0 | 5.9 | 33.7 | 1.90 | 47.7 | 3.6 |

## LARGE SCALE EXPERIMENTS, BOSTON MAIN DRAINAGE SYSTEM, 1936

Mention has been made herein of the use of one of the large storage tanks at Moon Island for continuous flow experiments to determine the removal of suspended solids and grease from the sewage of the Boston Main Drainage System. These experiments were conducted in the first place to compare, so far as practicable, the results of the small scale experiments with those of the large scale, and to give some indication of the possible use of the present equipment of the Boston Main Drainage System for purposes of sedimentation and grease removal. The tank used was trapezoidal in shape, 162 ft wide, and had an average length of 612 ft. The floor at the outer end, which is slightly lower than that of the inner end, was at Elevation 9 (mean sea level). The tank was equipped with a dike, with weir and skimming baffle and gates to prevent the sewage from flowing out of the bottom. The skimming baffle consisted of a floating, vertical, wooden structure about 3 ft deep, 2 ft of which were submerged, held 6 ft out in front and parallel to the sides of the dike by spreaders. The spreaders were connected in such a way as to allow the baffle to rise and fall with the level of the sewage. It was the purpose of the baffle to prevent the passage of this floating matter over the weir. During the first experiment, from August 11 to September 4, the settled sewage was stored in the three remaining tanks and discharged to sea in the regular manner on the second and third hours of the outgoing tide. Sewage from the experimental tank was discharged to sea for two hours longer to compensate for the loss of one tank from the total of the storage capacity. The average detention period in the tank was estimated to be about three hours. Beginning September 8, experiments were made in the same manner except that the sewage was discharged to sea

continuously; that is, the three storage tanks were not used, sewage passing through the experimental tank only. This part of the experiment was discontinued on September 18. At the conclusion of each week's run on Friday evening, the sluice gates were opened to allow the basin to drain and the lower section of each bulkhead was removed and the basin flushed out. Frequent cleaning of the tank prevented the occurrence of septic conditions with the formation of gas which would interfere with sedimentation. The scum was discharged to sea on the outgoing tide, as necessary. Samples of the raw sewage and the settled sewage were collected each half-hour and composited over an 8-hr period. Determinations for dissolved oxygen and pH were made in the field and the regular chemical determinations were made in the laboratory. The experiments were run continuously from Monday morning through Friday evening. Because the weir was submerged much of the time, accurate weir measurements of the flow could not be made, but estimates were made from the level of sewage in the basins before discharge. Considerable sewage, perhaps 5 to 10 mgd, escaped through the stop planks in the boat chamber, or through the numerous gates, and were not treated in the settling tank.

During continuous discharge, the weir readings were made every two hours and these measurements should show rather accurately the quantity of sewage treated in the settling tank; but they did not necessarily show the total flow to Moon Island, because of the losses mentioned. The large scale settling and skimming tests at Moon Island were made over a period of 28 operating days. During these experiments, harbor and shore samples were collected in addition to the regular composites of raw and settled sewage. Samples of the scum and sludge were also collected to determine their value for grease recovery and for fertilizer or fill.

The detention period varied from 1.7 to 5.8 hr and averaged 2.8 hr. During the test the average daily flow through the tank was about 60 million gal. The average results of the settling tests for the entire period are shown in Table 10.

TABLE 10.—SETTLING TESTS, MOON ISLAND; BOSTON MAIN DRAINAGE SYSTEM

| Average sewage concentration, in parts per million<br>(1) | Free ammonia<br>(2) | Chlorides<br>(3) | Oxygen consumed<br>(4) | Kjeldahl nitrogen<br>(5) | Fats<br>(6) | Bio-chemical oxygen demand<br>(7) | Suspended solids<br>(8) |
|---|---|---|---|---|---|---|---|
| Raw sewage | 18.1 | 3 478 | 49.6 | 8.58 | 34.2 | 144 | 119 |
| Settled sewage | 18.8 | 3 437 | 39.7 | 5.94 | 20.0 | 107 | 51.5 |
| Percentage reduction | .... | .... | 19.9 | 30.8 | 41.5 | 25.6 | 56.7 |

The raw sewage from the Boston Main Drainage System, during the tests of 1936, was somewhat weaker than during the experimental work in 1935 at Calf Pasture. The concentration of suspended solids was particularly less in 1936, averaging about 162 ppm in 1935 and 119 in 1936. The strength of the sewage varied from hour to hour and from day to day. The strongest sewage was received during the afternoon and early evening, and the weakest sewage

arrived in the early morning. The suspended solids in the raw sewage varied during the tests from 50 to 302 ppm. It is interesting to note that the removal of the material by sedimentation, in the large tank experiments, compared favorably with those obtained in the small unit on the sewage from this district at the Calf Pasture Pumping Station.

In connection with these experiments, it should be remembered that the large basin used for the experiment was designed as a storage tank and not as a settling tank, and accordingly the distribution of the sewage at the inlet was poor. The length of the effluent weir was small for the large volume of sewage, and the velocity of approach to the weir was relatively great. When a large volume of scum accumulated in the tank, the velocity of the sewage carried

FIG. 11.—VIEW OF MOON ISLAND, SHOWING IMPROVEMENT OVER CONDITIONS IN FIG. 3

some of the scum under the baffle and over the weir. Considering the fact that the large chamber was not well fitted for settling and skimming of sewage solids, the results obtained were very satisfactory and indicated that better results might be expected with a tank properly designed and operated.

It was impracticable to measure the sludge removed as it was distributed over the bottom of the tank in a thin layer, and it was impossible to draw down the tank without disturbing and losing some of the solids. However, calculations made from the reduction of the suspended solids indicate that, in all probability, an average of 21.2 tons of dry solids were removed daily, which compares favorably with the estimates obtained during the Calf Pasture Pumping Station experiments.

The quantity of floating matter held back on the surface of the sewage by the skimmers was much greater than expected, and rough estimations indicate that as much as 40 cu ft per million gal of sewage may be removed in this manner. The grease content of the sewage varied considerably from day to day. The average results of the analyses of the scum showed that it contained about 89% moisture and 77.5% volatile matter. An average of 42% of the dry solids was fat and 2.8% was Kjeldahl nitrogen.

The raw sewage had an average of 34.2 ppm of fat which was reduced to 20.0 ppm by sedimentation, a reduction of 41.5 per cent.

In order to determine the effect on the harbor waters, of the discharge of settled sewage, aerial photographs such as Fig. 11 were taken during the large-scale experiments at Moon Island. A comparison with Fig. 3, taken under conditions of normal discharge, indicates that the sleek areas resulting from the discharge of settled sewage were not as sharply defined as before and that the appearance of the harbor water was considerably improved. Conditions on the incoming tides during periods of continuous discharge at Moon Island, however, were not sufficiently satisfactory to make such a practice advisable unless a new point of discharge more remote from the shore is provided.

TABLE 11.—VIABILITY OF SEWAGE BACTERIA IN SEA WATER

| Time, in hours from the start | Bacteria per Cubic Centimeter | | | *Coli-Aerogenes* per cubic centimeter | Bacteria per Cubic Centimeter | | | *Coli-Aerogenes* per cubic centimeter |
|---|---|---|---|---|---|---|---|---|
| | Four days, 20° Centigrade | Twenty-four Hours, 37° Centigrade | | | Four days, 20° Centigrade | Twenty-four Hours, 37° Centigrade | | |
| | | Total | Reduction | | | Total | Reduction | |
| | (*a*) SEWAGE | | | | (*e*) TAP WATER, WITH 1½ PER CENT SEWAGE | | | |
| 0 | 1 200 000 | 270 000 | 130 000 | 20 000 | 120 000 | 3 800 | 2 400 | 2 000 |
| 24 | 2 200 000 | 340 000 | 195 000 | 10 000 | 230 000 | 3 100 | 1 900 | 1 000 |
| 48 | 1 000 000 | 120 000 | 50 000 | 10 000 | 260 000 | 2 800 | 1 200 | 1 000 |
| 96 | 1 800 000 | 22 000 | 9 000 | 5 000 | 16 000 | 1 100 | 630 | 1 000 |
| | (*b*) SEA WATER | | | | (*f*) SEA WATER, WITH ½ PER CENT SEWAGE | | | |
| 0 | 340 | 1 | 0 | 0 | 12 000 | 1 400 | 400 | 100 |
| 24 | 400 | 1 | 0 | 0 | 2 500 | 110 | 40 | 100 |
| 48 | 590 | 1 | 0 | 0 | 2 400 | 110 | 50 | 20 |
| 96 | 480 | 1 | 0 | 0 | 2 000 | 8 | 1 | 0* |
| | (*c*) TAP WATER, WITH ½ PER CENT SEWAGE | | | | (*g*) SEA WATER, WITH 1 PER CENT SEWAGE | | | |
| 0 | 48 000 | 1 800 | 800 | 500 | 22 000 | 2 100 | 700 | 500 |
| 24 | 380 000 | 1 400 | 600 | 500 | 4 000 | 140 | 70 | 100 |
| 48 | 180 000 | 1 600 | 1 000 | 1 000 | 3 300 | 150 | 60 | 20 |
| 96 | 26 000 | 850 | 420 | 500 | 1 800 | 5 | 0 | 0* |
| | (*d*) TAP WATER, WITH 1 PER CENT SEWAGE | | | | (*h*) SEA WATER, WITH 1½ PER CENT SEWAGE | | | |
| 0 | 84 000 | 2 700 | 1 200 | 1 000 | 36 000 | 4 000 | 1 400 | 1 000 |
| 24 | 480 000 | 2 500 | 1 400 | 500 | 3 200 | 180 | 90 | 100 |
| 48 | 270 000 | 1 800 | 1 100 | 1 000 | 3 900 | 130 | 60 | 50 |
| 96 | 21 000 | 1 300 | 900 | 1 000 | 2 300 | 6 | 1 | 0* |

* *Coli-Aerogenes* present in 10 cubic centimeters.

## Viability of Sewage Organisms in Salt Water

Studies made by the Massachusetts Department of Public Health to determine the effect of polluted water on quahaugs, a mollusk of the hard-shell variety, showed that quahaugs obtained from polluted waters were comparatively free from organisms of the *Coli-Aerogenes* group after immersion for 48 hr in clean sea water. In order to ascertain if the disappearance of organisms of this group was due to the shellfish themselves or to the sea water, or both, experiments were conducted at the Lawrence Experiment Station of the Department of Public Health to determine the reduction of bacteria of the *Coli-Aerogenes* group in sewage when discharged into sea water. Similar experiments also were made to show the effect on these bacteria when discharged into the water drawn from a tap of the city of Lawrence. In these experiments, sewage was added to the sea water and the tap water in proportions of ½%, 1% and 1½% of the volume of water into which the sewage was added, and the samples were kept shielded from direct sunlight, the temperature varying from 50° to 70° F. The sewage alone and the untreated sea water were kept under the same conditions and analyses were made after 1, 2, and 4 days. The results of these tests (see Table 11) show that the total

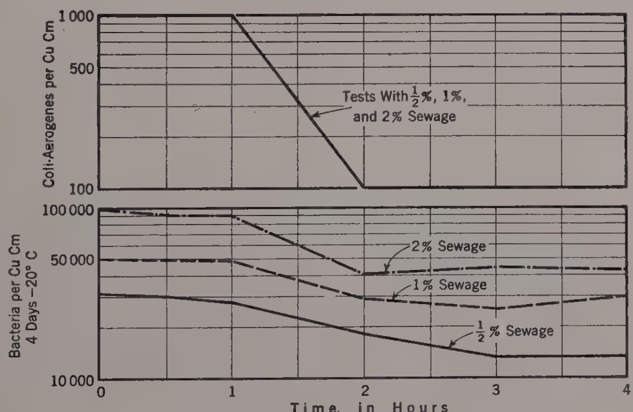


Fig. 12.—Viability of Bacteria in Sea Water

bacteria in the samples incubated at 20° and 37° C, as well as the organisms of the *Coli-Aerogenes* group, decreased very slowly in the sewage and the tap-water mixture, but very rapidly in the sewage and sea-water mixture. Because of this very considerable reduction in the numbers of bacteria in some instances, in the first 24 hr, the experiments were later continued by providing shorter periods of contact of the sewage with both the fresh and salt water. The latter results are shown on Fig. 12, a comparison between regular sewage and Nahant sea water being as follows:

| Sample | Number of bacteria per cubic centimeter | Number of *Coli-Aerogenes* per cubic centimeter |
|---|---|---|
| Regular sewage | 5 300 000 | 100 000 |
| Nahant sea water | 770 | 1 |

## Remedial Measures Considered for Alleviating Pollution of Boston Harbor

Although the discharge of sewage into Boston Harbor has not as yet caused conditions dangerous to the public health, objectionable floating matters characteristic of sewage may sometimes reach certain shores and, in the future, the demands of the public may require improvement.

To eliminate the pollution of Boston Harbor, either the sewage may be: (1) Discharged farther out to sea; (2) treated for the removal of suspended solids and grease before its discharge into the harbor; or, (3) treated to remove the solids and then conveyed by tunnel and discharged outside the harbor. Consideration was given to the construction of an outfall sewer which would intercept the sewage from the existing main sewer outlets—namely, the North Metropolitan outlet at Deer Island, the outlet of the Boston Main Drainage District at Moon Island, and that of the South Metropolitan outlet at Nut Island—and which would have a point of outfall in the Outer Harbor in the vicinity of The Graves. It was found to be impracticable to construct such an intercepting outfall sewer unless it was in the form of a tunnel, because Boston Harbor consists of a number of deep valleys filled with clay and other glacial deposits. Between these valleys are peaks of ledge which reach in some places above mean high water. Accordingly, with the aid of a geologist, the Waterways Division of the Massachusetts Department of Public Works prepared estimates of the cost of constructing this outfall sewer in the form of a tunnel in ledge. The engineering and geological part of the investigation showed that the sewage from the Boston Main Drainage District can be disposed of by a tunnel 9 ft in diameter from Moon Island to Rainsford Island (a distance of 9 950 ft); and that the sewage from the South Metropolitan District can be sent in a tunnel 15 ft in diameter in rock from Nut Island to Rainsford Island (a distance of 10 450 ft). The tunnels from Moon Island and Nut Island would form a joint tunnel at Rainsford Island, 18 ft in diameter, also in rock, which would extend to Lovell Island, a distance of 10 000 ft. At Lovell Island this tunnel would intercept an outfall sewer from Deer Island which would be for the most part in glacial deposit and would be 14 ft in diameter and 9 200 ft long. The sewage from the Deer Island connection would enter the tunnel by way of a shaft. From Lovell Island the tunnel would be 23 ft in diameter and would extend to the vicinity of The Graves, a distance of 18 600 ft. The latter tunnel also would be entirely in rock and would have a capacity of 1 205 million gal in 24 hr. According to the design proposed, provision would be made to discharge the sewage at The Graves through a system of multiple outlets where the depth of water at mean low tide is 30 to 35 ft. The estimated capitalized costs of these outlet works, including the radial outlets from the terminal chamber and all tunnel construction, but not including the cost of new pumping facilities, were $26 100 000. The capacity of these works which was for a flow of sewage amounting to 1 205 mgd is approximately five times the average daily flow of sewage from these districts in 1935.

Chips that were released in the vicinity of The Graves reached the shores of Boston Harbor and various points along the coast remote from The Graves,

proving rather conclusively that, if the sewage is to be prevented from reaching the shores and causing objectionable conditions among bathers, it will be necessary, even with the outlet as remote as at The Graves, to treat it for the removal of grease and floating matter. Under the circumstances, the plan of extending the sewer outlets to the vicinity of The Graves would not appear to be justified until it is necessary to provide such works in connection with the disposal of treated sewage. Fig. 1 shows the location of the suggested outfall works to The Graves.

The most economical plan appeared to be to provide treatment works in the vicinity of the main sewer outlets at Deer Island, Moon Island, and Nut Island. When the treated sewage discharged from such works causes objectionable conditions in the harbor a program of progressive tunnel development can be provided which will permit the treated sewage to be discharged, in the beginning, in the vicinity of Rainsford Island. Float tests indicated that, near this island, sewage treated for the removal of grease and suspended wastes could be dissipated on either the incoming or outgoing tides before reaching any of the main ship channels.

Treatment works for the removal of suspended solids and grease would consist of bar screens and sedimentation tanks with skimming and sludge removal equipment. The skimming and sludge would be disposed of by barging to sea. The first cost of such works for the three districts, including the cost of operation and the cost of disposing of the sludge, grease, and other floating matter, together with the cost of additional pumping, is shown in Table 12.

TABLE 12.—First Cost of Treatment Works

| Description | Boston Main Drainage System | North Metropolitan Sewerage System | South Metropolitan Sewerage System |
|---|---|---|---|
| First cost of works consisting of bar screen with mechanical cleaning devices, sedimentation tanks with skimming and sludge removal equipment | $2 907 000 | $ 5 539 850 | $ 6 023 100 |
| Capitalized cost of operation and disposal of skimmings, screenings, and sludge by barging to sea, to 1955* | 3 300 000 | 7 190 000 | 7 560 000 |
| Annual charges | (123 630) | (269 910) | (283 290) |
| Total cost of partial treatment to 1955 | $6 207 000 | $12 729 850 | $13 583 000 |

* Interest at 3¾ per cent.

## Sludge Disposal

The problem of sludge disposal is usually the most difficult one to be solved in the treatment of sewage. The location of the proposed plants on islands in Boston Harbor points at once to the disposal of sewage sludge by barging to sea; and this would undoubtedly be the most economical method, especially if the sludge could be carried to sea as cheaply as is done by the Passaic Valley (N. J.) Sewerage District where the cost is only 14.7 cents per wet ton. In Boston, operation of sludge steamers or barges by the authorities in charge of sewage disposal would probably be less expensive than a contract, as sea-going ships of the type necessary for this work are not now available in Boston.

Representative samples of sludge from the three systems were analyzed by the Massachusetts Agricultural Experiment Station and the organization reported that the fat content is rather high and that the product is rather poorly balanced as a fertilizer for most crops, as it contains too much nitrogen as compared with the phosphoric acid and potash. For many hoed crops the product used at the rate of 1.5 tons per acre would be properly balanced by the use of about 1 200 to 1 400 lb of 16% super-phosphate and 240 to 300 lb of muriate of potash per acre.

Although the park departments of the city of Boston and the Metropolitan District Commission were not interested in the use of sewage sludge as fertilizer, provision should be made in designing the treatment plants for proper elevations and sufficient area for the preparation of sludge as fertilizer if it should become desirable.

## Summary

This study shows that the quantity of putrescible matter discharged into the harbor is not sufficient as yet to cause a nuisance resulting from the depletion of oxygen. Objectionable sleek areas, however, are often present over extensive portions of the harbor and, under certain tide and wind conditions, grease and other floating matters, characteristic of sewage, reach the shores. At times of considerable run-off, sewage overflows from the main trunk and lateral sewers and reaches shores used for bathing.

Bacterial examinations do not show the presence of bacteria characteristic of sewage pollution in the waters of the bathing beaches in such numbers as would warrant the prevention of bathing except in the vicinity of sewer overflows and at a few places where sewage is discharged directly into the harbor.

To correct these unsatisfactory conditions it will be necessary to discontinue the use of certain areas as bathing beaches or to make changes in the sewerage systems by moving the sewer overflows to points remote from the shore and by treating the sewage reaching the main outlets for the removal of matters that might reach the bathing places.

Although the discharge of sewage into Boston Harbor from the main sewer outlets may not as yet cause conditions dangerous to the public health, the nuisance from sleek and floating solids makes a change in the present methods of sewage disposal very desirable.

The most economical method seems to be the removal of suspended and floating solids in sedimentation tanks provided with skimming devices and the transportation of the solids and grease to sea in barges. More complete treatment and the extension of the present outlets at some future time, when larger sewage flow makes it necessary, are the logical steps in the disposal of the sewage of Metropolitan Boston.

The extension of the outlet systems by a tunnel to a site in the vicinity of The Graves without preliminary treatment of the sewage seems impracticable because of the high cost and because floating solids would probably still reach the neighboring shores.

AMERICAN SOCIETY OF CIVIL ENGINEERS

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PAPERS

# DESIGN OF CIRCULAR CONCRETE TANKS

By George S. Salter,[1] M. Am. Soc. C. E.

## Synopsis

The object of this paper is to present a rigorous mathematical design analysis of circular concrete tanks that is accurate for shallow tanks of rather large diameters, such as are used for settling purposes, as well as for deeper tanks of small diameter. It is shown that, by fixing the wall at the base so that the liquid pressure is divided between the circular rings and the vertical cantilevers, the magnitude of the maximum ring tension is greatly reduced. For tanks of proportions such as are ordinarily used for settling purposes the maximum ring tension amounts to only about one-quarter to one-third, and the total amount of wall reinforcement to less than one-half of that required were the wall free at the base. Diagrams are presented to facilitate design and an example is given showing their application.

## Introduction

Theoretically, circular tanks having the least perimeter for a given area are the most economical that can be designed. If no restraint were offered at the junction of the wall with the base, the internal liquid pressure would resolve itself into pure tension and the ordinary cylinder theory would obtain for the full height of the wall. It has been quite common practice to design circular tanks for this condition and (although the wall is rigidly connected to the bottom slab) to provide ring reinforcement for the full liquid pressure. As concrete is a material that lends itself inherently to continuity, a logical method of design using this material is one that takes advantage of this action. Circular tanks built of concrete, however, have more or less restraint at the junction of the wall and the footing unless special precautions are taken to eliminate this action.

If the tank wall were rigidly fixed at the base, the ring action would be eliminated at the point where the pressure is the greatest, and thus the percentage of ring steel could be reduced; but this restraint of the wall at the base

Note.—Written comments are invited for immediate publication; to insure publication the last discussion should be submitted by **May 15, 1939**.

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leads to rather complicated load distribution between the circular ring elements and the vertical cantilever elements. The division of the load between these two structural elements is emphasized by the disposition of the reinforcement in the wall. The size and spacing of the vertical bars is determined by the wall moments, due to that portion of the load carried vertically; and the horizontal ring steel is determined by the magnitude of the ring tension. Naturally, the sum of the loads carried vertically and horizontally must equal the total load and pressure on the wall.

Several methods have been presented in endeavors to evaluate the load distribution between the two structural elements but they have either been merely approximations or have been quite difficult to apply. Furthermore, most previous studies, apparently, were based on the condition of rather deep tanks or tanks of small diameter and thus, although they gave approximately correct results for tanks of those proportions, they were decidedly "out of line" for large-diameter, and rather shallow, tanks such as are ordinarily used for settling purposes for sewage, water purification, and certain metallurgical processes.

The purpose of this paper is to present a strictly rigorous analysis and yet one that is easy to apply and which yields results that are accurate, within the limits of the assumptions made, for tanks of any proportions. Although the analysis applies to tanks of all sizes, the diagrams presented herein apply only to tanks of proportions such as are ordinarily used for settling purposes. Of course, these diagrams can be extended to include tanks of all sizes.

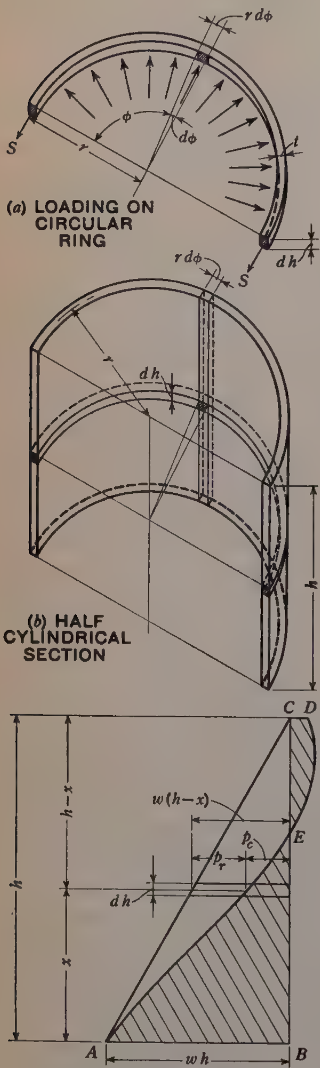

FIG. 1

## NOTATION

The letter symbols used in this paper are defined where they are first mentioned. An effort has been made to conform to the Symbols for Mechanics, Structural Engineering, and Testing Materials[2] approved by a committee of the American Standards Association, with Society representation, in 1932.

## ANALYSIS

The following is presented as a rigorous analysis of the problem when the wall is considered as an elastic cylinder, fixed at the base. Given a tank of

[2] A. S. A.—Z10a—1932.

radius $r$ and height $h$. With origin at the base of the wall, the internal pressure at any point, due to a liquid whose unit weight is $w$, is $w\,(h - x)$, in which $x$ is the distance above the base of the wall.

Consider one-half of a cylindrical tank as represented in Fig. 1(*a*). The vertical strip of height $h$ and width $r\,d\phi$ represents a vertical cantilever element, and the horizontal element of height $dh$ represents one-half of a circular ring element. If the wall is fixed at the base the liquid pressure will be resisted in part by cantilever action and in part by ring action. The distribution of the load between the two elements must be such as to make the deflection of the cantilever equal to the radial displacement of the ring.

Consider first the loading on the circular ring. Fig. 1(*a*) represents one-half of a ring element of height $dh$, and thickness $t$. It is subjected to a uniform radial load, which is some part of the liquid pressure, of intensity $p_r$, and is assumed to be acting on the center line of the ring. Furthermore, the thickness is assumed to be so small compared to the radius that the stress on the cross-sectional area is uniform.

The force acting on an element of the ring is $p_r\,r\,d\phi\,dh$, in which $d\phi$ is the central angle that subtends the vertical strip. Taking the sum of the components of all the radial forces acting on the half ring, the equation of equilibrium is as follows:

$$2\,S = 2\int_0^{0.5\pi} p_r \sin\phi\; r\,d\phi\,dh = 2\,p_r\,r\,dh \qquad (1)$$

and, therefore,

$$S = p_r\,r\,dh \qquad (2)$$

The unit tensile stress is found by dividing the total stress by the cross-sectional area of the ring:

$$f_s = \frac{p_r\,r\,dh}{t\,dh} = \frac{p_r\,r}{t} \qquad (3)$$

For a given modulus of elasticity, $E_0$, of the shell, the elongation becomes $\frac{2\,\pi\,r\,f_s}{E_0}$ and the resultant increase in the radius, denoted by $y$, is

$$y = \frac{2\,\pi\,r\,f_s}{2\,E_0\,\pi} = \frac{r\,f_s}{E_0} \qquad (4)$$

Substituting the value of $f_s$ given in Equation (3)

$$y = \frac{p_r\,r^2}{E_0\,t} \qquad (5)$$

and, therefore,

$$p_r = \frac{E_0\,t}{r^2}\,y \qquad (6)$$

Next, consider a vertical strip, of width $r\,d\phi$ and height $h$, also as indicated in Fig. 1(*b*). The total liquid pressure is represented by the triangle in Fig. 1(*c*) and this is divided into two parts by the "load-distribution curve" $A\,E\,D$. The cross-hatched area represents that part of the load resisted by the canti-

lever, and the algebraic sum of the loads on the cantilever and the ring must equal the liquid pressure; that is,

$$p_c + p_r = w\,(h - x) \quad \text{.....} \quad (7a)$$

or

$$p_c = w\,(h - x) - p_r \quad \text{.....} \quad (7b)$$

Substituting the value of $p_r$, as given in Equation (6), in Equation (7*b*), the load on the cantilever may be expressed as

$$p_c = w\,(h - x) - \frac{E_0\,t}{r^2}\,y \quad \text{.....} \quad (8)$$

The general differential equation of the elastic curve of a beam under any system of continuous loading is expressed[3] by

$$E\,I\,\frac{d^2y}{dx^2} = M \quad \text{.....} \quad (9a)$$

in which $M$ = moment at any point; also,

$$E\,I\,\frac{dy}{dx} = \theta \quad \text{.....} \quad (9b)$$

$$E\,I\,\frac{d^3y}{dx^3} = V \quad \text{.....} \quad (9c)$$

and

$$E\,I\,\frac{d^4y}{dx^4} = p_c \quad \text{.....} \quad (9d)$$

in which $\theta$, $V$, and $p_c$ equal slope, shear, and unit load, respectively. Substituting in Equation (9*d*) the value of $p_c$ given in Equation (8)

$$E\,I\,\frac{d^4y}{dx^4} = w\,(h - x) - \frac{E_0\,t}{r^2}\,y \quad \text{.....} \quad (10a)$$

or

$$\frac{d^4y}{dx^4} + \frac{E_0\,t}{r^2\,E\,I}\,y = \frac{w\,(h - x)}{E\,I} \quad \text{.....} \quad (10b)$$

The general solution of Equation (10*b*) is[4]

$$y = e^{\beta x}\,(A \cos \beta\,x + B \sin \beta\,x) + e^{-\beta x}\,(C \cos \beta\,x + D \sin \beta\,x) + \frac{w\,(h - x)\,r^2}{E_0\,t} \quad \text{.....} \quad (11)$$

in which $A$, $B$, $C$, and $D$ are arbitrary constants whose values must be determined from known conditions at certain points; $e$ is the base of Naperian logarithms; $E_0$ = modulus of elasticity of the reinforced circular rings,

[3] "Strength of Materials," by S. Timoshenko, Part I, p. 182.

[4] *Loc. cit.*, Part II, p. 402.

$E$ = modulus of elasticity of the reinforced vertical cantilevers; $I$ = moment of inertia of the vertical section $= \frac{t^3}{12}$; and,

$$\beta = \sqrt[4]{\frac{E_0\,t}{4\,r^2\,E\,I}} \quad \text{.......................(12)}$$

By successive differentiation of Equation (11) the following equations may be secured:

$$\frac{dy}{dx} = \beta\,e^{\beta x}\,[(A + B)\cos\beta x - (A - B)\sin\beta x]$$
$$- \beta\,e^{-\beta x}\,[(C - D)\cos\beta x + (C + D)\sin\beta x] - \frac{w\,r^2}{E_0\,t} \quad \text{.....(13a)}$$

$$\frac{d^2y}{dx^2} = 2\,\beta^2\,e^{\beta x}\,(B\cos\beta x - A\sin\beta x)$$
$$- 2\,\beta^2\,e^{-\beta x}\,(D\cos\beta x - C\sin\beta x) \quad \text{..............(13b)}$$

$$\frac{d^3y}{dx^3} = -\,2\,\beta^3\,e^{\beta x}\,[(A - B)\cos\beta x + (A + B)\sin\beta x]$$
$$+ 2\,\beta^3\,e^{-\beta x}\,[(C + D)\cos\beta x - (C - D)\sin\beta x] \quad \text{........(13c)}$$

and,

$$\frac{d^4y}{dx^4} = -\,4\,\beta^4\,e^{\beta x}\,(A\cos\beta x + B\sin\beta x)$$
$$- 4\,\beta^4\,e^{-\beta x}\,(C\cos\beta x + D\sin\beta x) \quad \text{..............(13d)}$$

Under the condition of base fixed and top free, the slope and deflection are zero at the base of the wall where $x = 0$, whereas, at the top of the wall where $x = h$, the moment and shear are zero. Thus, by setting, for the proper values of $x$, the appropriate one of Equations (13) equal to zero, and solving, the values of the four arbitrary constants may be determined, and are expressed as follows:

$$A = \frac{w\,r^2}{\beta\,E_0\,t}\left\{\frac{e^{2\theta}\,[2\cos^2\theta + \theta\,(\sin 2\,\theta + \cos 2\,\theta - 4\cos^2\theta)] - \theta}{(1 + e^{2\theta})^2 + 4\,e^{2\theta}\cos^2\theta}\right\} = F\,C_1 \quad \text{...(14a)}$$

$$B = \frac{w\,r^2}{\beta\,E_0\,t}\left\{\frac{1 + \theta + e^{2\theta}\,[1 + \sin 2\,\theta - \theta\,(\cos 2\,\theta + \sin 2\,\theta)]}{(1 + e^{2\theta})^2 + 4\,e^{2\theta}\cos^2\theta}\right\} = F\,C_2 \quad \text{...(14b)}$$

$$C = -\,F\,(C_1 + \theta) \quad \text{.....................(14c)}$$

and,

$$D = F\,(1 - \theta - 2\,C_1 - C_2) \quad \text{................(14d)}$$

in which,

$$\theta = \beta\,h \quad \text{..........................(15a)}$$

and,

$$F = \frac{w\,r^2}{\beta\,E_0\,t} \quad \text{..........................(15b)}$$

It may be readily seen that there is only one variable in Equations (14) so that for any given value of this term the expression in brackets has a definite

value. Thus, by allotting values to $\theta$ at regular intervals, it is best to express these values in terms of $\pi$ so as to facilitate computations; the values of the arbitrary constants may be determined, and thus by substitution in Equations (13*b*), (13*c*), and (13*d*), the moment, shear, and load, for any value of $\theta$, at any wall height, may be determined. In Fig. 2, Functions $C_1$ and $C_2$ are shown graphically.

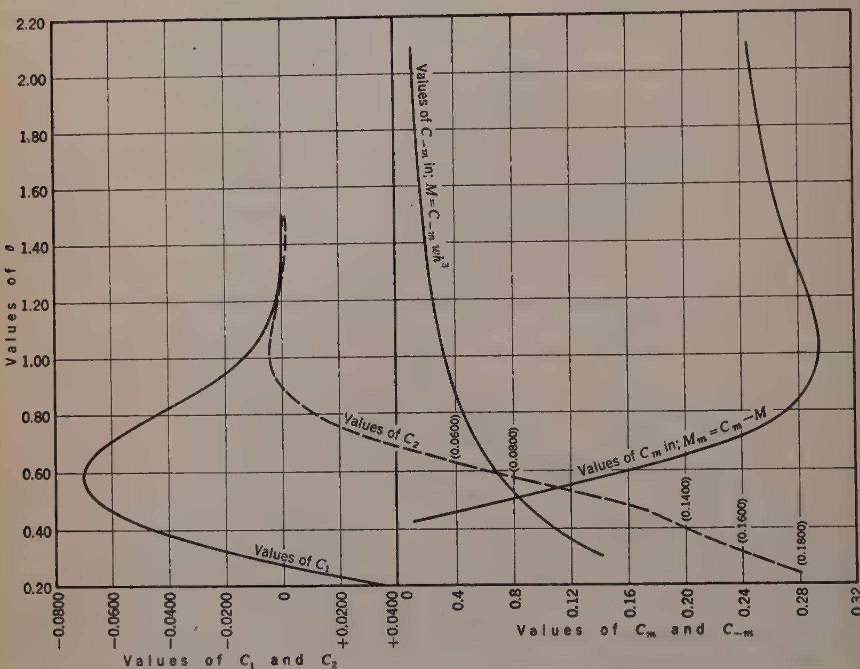


FIG. 2.—VALUES OF ARBITRARY CONSTANTS IN EQUATIONS (13) AND (14)

FIG. 3.—COEFFICIENTS FOR USE IN DETERMINING MOMENTS IN A CANTILEVER WALL SECTION

## PREPARATION OF DIAGRAMS

Four values are essential in the design of the tank wall: (1) The moment, (2) shear at the base of the cantilever elements, (3) the tension in the ring elements, and (4) the wall moment, at some point above the base and of opposite sign to that at the base, due to the upper ring elements supporting the cantilevers.

By the use of Equation (13*b*) curves were plotted for the various values of $\theta$, by which the moment at any height of the wall could be determined. However, as the moments at only two points are needed in the design, the negative moment at the base, and the positive moment at some point above the base, those curves were used to prepare the curves given in Fig. 3. In Fig. 3 the value of $C_{-m}$, cantilever negative moment coefficient, increases for diminishing values of $\theta$, becoming 0.167 when $\theta$ becomes 0, the limiting case when the wall is a pure cantilever. The curve for $C_m$ in Fig. 3 shows that the maximum value of the ratio of the positive moment to the negative moment occurs for

a $\theta$-value of approximately 1.0 $\pi$, and amounts to about 29.4% of the negative moment. This percentage decreases rapidly for $\theta$-values below 1.0 $\pi$ and less rapidly for $\theta$-values above 1.0 $\pi$.

Fig. 4 gives the "load-distribution curves" for values of $\theta$ from 1.00 $\pi$ to 2.00 $\pi$. The curves give the load carried by the circular rings, expressed as a percentage of the pressure at the base of the wall. These values were determined for the values of $\theta$ indicated by finding the load on the cantilever

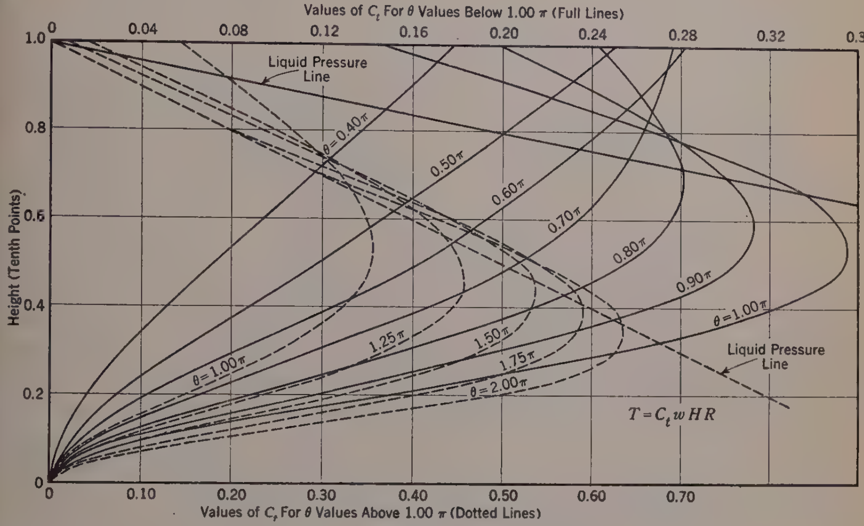


FIG. 4.—TENSION IN CIRCULAR RINGS

using Equation (13*d*) and deducting that part of the load from the total load at any point and expressing the remainder as a percentage of the load at the base of the wall. The form of the curve is determined by the relation $\frac{h^2}{r\,t}$. All tanks with equivalent values of $\theta$ have similar load-distribution curves.

It will be noted that, for all values of $\theta$, the percentage of the load carried by the rings at the base of the wall is zero. This indicates that the pressure at this point is resisted entirely by the cantilevers. In the upper parts of the wall, for the lower values of $\theta$, the load on the rings exceeds the liquid pressure. This shows that there is negative pressure on the cantilevers; or, in other words, the cantilevers are supported by the rings. This obtains from the fact that if the cantilevers were not supported at this elevation they would have their maximum deflection at this point, and the rings would not deflect at all, because the water pressure at the surface is zero. According to the assumptions of this paper the displacements of the rings and the cantilevers at all points are equal, and thus the rings at the higher elevations support not only the direct liquid pressure but also some load coming to them from the vertical cantilever elements. For the higher values of $\theta$, the load on the rings in the upper parts of the wall is practically equal to the liquid pressure, as the curves

show. This indicates that for the deeper tanks, or for those of smaller diameter, restraint at the base has little or no effect at the upper parts of the wall, as would be expected.

The shear is determined by use of Equation (13*c*). The coefficient for determining its value has been plotted in Fig. 5 and for ease of application has been expressed in percentages of the total load; thus, the shear at the base of the wall is found by the equation

$$V = C_v \frac{w\,h^2}{2} \quad \ldots\ldots\ldots(16)$$

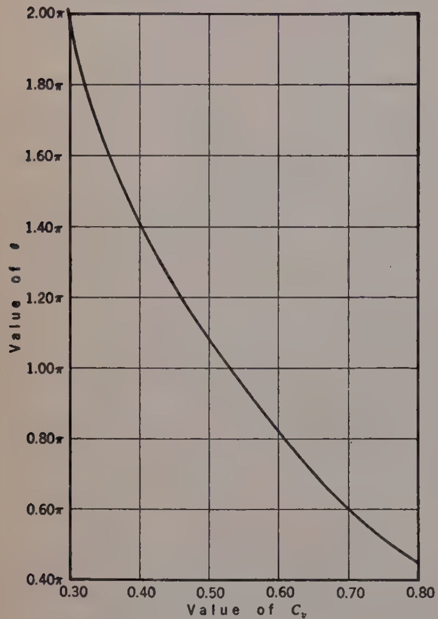


FIG. 5.—COEFFICIENTS FOR USE IN DETERMINING SHEAR AT BASE OF WALL

Although computed by use of Equation (13*c*) its value may be checked by determining, from Fig. 4, values of $C_m$ for the load on the ring elements and deducting this load from the total liquid pressure. It will be noted that, for a $\theta$-value of about 1.08 $\pi$, the total load is divided equally between the ring elements and the cantilever elements. For $\theta$-values less than 1.08 $\pi$ (which will include practically all shallow tanks of large diameter) the load on the cantilever will be in excess of 50% of the total load, and thus the ring tension will be less than 50% and the total ring reinforcement required will be less than one-half of that required if all of the load were carried by the rings.

It is desirable to have a simplified expression to determine the value of $\theta$ for use in connection with the diagrams and this may be found as follows:

Substituting, in Equation (12), $\frac{t^3}{12}$ for $I$ and assuming that $E_0 = E$, which is approximately correct for most tanks for which the diagrams are applicable, the expression for $\theta$ may be given as,

$$\theta = \frac{h}{\pi}\sqrt[4]{\frac{3}{r^2\,t^2}} = h\sqrt{\frac{0.176}{r\,t}} \quad \ldots\ldots\ldots\ldots\ldots\ldots(17)$$

Note that the value of $\theta$ is expressed in terms of $\pi$ and it is so indicated on the diagrams.

## EXAMPLE

The use of the diagrams in design will be demonstrated by a numerical example. Certain conditions have been assumed in the analysis, and the designer should consider how closely the actual conditions fulfill these requirements. The most important are: The degree of restraint at the base of the

wall; the moment of inertia of the reinforced section; the modulus of elasticity of the concrete; and the extent to which the concrete itself resists tension.

As in most structures where water-tightness is required, a rather rich concrete should be specified, and to reduce cracking, a low stress in the ring steel should be used; 12 000 lb per sq in. is recommended. The percentage of the ring reinforcement should be limited so that the tensile stress in the concrete, caused by combined direct tension and contraction due to curing and drying, will not exceed 200 to 250 lb per sq in. In order to secure better values in bond and also to distribute the stresses properly (both of which will help to eliminate cracking), rather small bars, preferably those not exceeding ⅝ in. in diameter, should be used.

Given a tank 70 ft in diameter and a water depth of 12.5 ft, assume a wall thickness of 9 in.; by Equation (17), $\theta = 12.5\sqrt{\dfrac{0.176}{35.4 \times 0.75}} = 1.018\,\pi$. From Fig. 3, $C_{-m} = 0.0329$; therefore $-M = 0.0329 \times 62.5 \times (12.5)^3 = 4\,020$ ft-lb per ft. Furthermore (see Fig. 3), $C_m = 0.294$; therefore $M = 0.294 \times 4\,020 = 1\,180$ ft-lb per ft.

The vertical wall reinforcement is determined from these moments. It is sufficient to determine, from Fig. 4, the values of the ring tension coefficients at the tenth-points, and thus compute the percentage of ring steel for the full height of the wall. It will be noted in Table 1 that the maximum ring tension is at about the mid-point of the wall; that is, $T_{\max} = 0.364 \times 62.5 \times 12.5 \times 35.4 = 10\,050$ lb; and, $A_s = \dfrac{10\,050}{12\,000} = 0.84$ sq in. per ft of wall height. From Fig. 5, $C_v = 0.523$; therefore, $V = 0.523\,\dfrac{62.5 \times (12.5)^2}{2} = 2\,560$ lb per ft.

TABLE 1.—COMPUTATION OF RING STEEL

| Tenth-points with Point 1.0 at the top (1) | Coefficient $C_t$ (2) | Ring tension, in pounds per foot of wall height (3) | Area of steel, $A_s$, in square inches per foot of height (4) | Tenth-points with Point 1.0 at the top (1) | Coefficient $C_t$ (2) | Ring tension, in pounds per foot of wall height (3) | Area of steel, $A_s$, in square inches per foot of height (4) |
|---|---|---|---|---|---|---|---|
| 1.0 | 0.137 | 3 780 | 0.32 | 0.5 | 0.364 | 10 050 | 0.84 |
| 0.9 | 0.204 | 5 640 | 0.47 | 0.4 | 0.334 | 9 220 | 0.77 |
| 0.8 | 0.266 | 7 350 | 0.62 | 0.3 | 0.262 | 7 240 | 0.61 |
| 0.7 | 0.322 | 8 890 | 0.74 | 0.2 | 0.158 | 4 360 | 0.37 |
| 0.6 | 0.356 | 9 820 | 0.84 | 0.1 | 0.053 | 1 460 | 0.12 |

The wall should be investigated for external pressures in the same manner. For earth loading this may be done by substituting for the earth pressure an equivalent fluid; and, if the external loading extends to either a greater or less height than the internal loading, also substitute the revised value for the height. For external loading the rings are in compression and thus do not require reinforcement.

The wall thickness should be checked to make certain that the tension stress in the concrete is not excessive. The curing stress may be determined

from the usual formula.[5]

$$f_c = \frac{C\,E_c\,n\,p}{1 + n\,p} \qquad \text{(18)}$$

in which $C$ = coefficient of contraction of the concrete, assumed as 0.0003. Use the transformed section to find the stress in the concrete due to the ring tension:

$$f_t = \frac{T}{A_c\,(1 + n\,p)} \qquad \text{(19)}$$

The sum of $f_c$ and $f_t$ should not exceed 200 to 250 lb per sq in.:

$$f_c = \frac{0.0003 \times 2\,500\,000 \times 12 \times 0.0077}{1 + 12 \times 0.0077} = 64$$

and

$$f_t = \frac{10\,050}{9 \times 12\,(1 + 12 \times 0.0077)} = 85$$

and, total stress, in pounds per square inch = 149.

The 9-in. wall is adequate. Note that 0.0077 is the maximum percentage of the ring reinforcement; that is, $p = \frac{0.84}{9 \times 12} = 0.0077$.

## Conclusion

It is intended that this paper should provide a simple and direct method of designing circular concrete tanks. The writer feels that the accuracy of the rigorous analysis has been retained in the diagrams presented and that by their use circular concrete tanks can be designed, not only more easily, but also more economically, than with the approximate or rule-of-thumb methods now commonly in use.

## Acknowledgment

The paper is based upon a thesis entitled "The Design of Circular Concrete Tanks," presented by the writer to the University of Nebraska in 1937, in partial fulfillment of the requirements for the professional degree of Civil Engineer.

[5] "Principles of Reinforced Concrete Construction," by F. E. Turneaure, Hon. M. Am. Soc. C. E., and Ward B. Maurer, Assoc. M. Am. Soc. C. E., Third Edition, p. 36.

AMERICAN SOCIETY OF CIVIL ENGINEERS

Founded November 5, 1852

REPORTS

# WATER SUPPLY ENGINEERING[1]

## PROGRESS REPORT OF COMMITTEE OF SANITARY ENGINEERING DIVISION

This annual report covers such matters relating to noteworthy water supply projects—under construction or projected—catastrophes involving water supplies, and progress in the art of water supply, as have come to the notice of members of the Committee or its correspondents who have kindly aided the Committee.

### Water Works Construction

*Metropolitan Water District of Southern California.*—This mighty project for damming the Colorado River about 125 miles below Boulder Dam and conveying it to Los Angeles and vicinity through an aqueduct, the main part of which is about 250 miles long with large branches totaling about 140 miles in addition, is approaching the stage where the first use may be begun. Parker Dam, an arch dam 320 ft high above foundation (only about 75 ft above river bed) on the Colorado River, has been completed and water passed through its gates. Early in January the first water was pumped from the dam to the first booster station of Gene Reservoir about two miles from the dam. The last of twenty-nine tunnels having a total length of 92 miles along the 250-mile aqueduct was "holed through" on November 19, 1938, and the aqueduct was reported complete in that month.[2] It is reported that plans are in preparation for softening the Colorado River water. The softening plant which is being studied would not be at the source but on the Foothill Boulevard branch of District's distribution system from which point it would be delivered to each of the district cities. The raw water has a hardness of about 250 ppm and total solids of about 500 ppm.

*Central Valley Project, California.*—This $170 000 000 project of the U. S. Reclamation Service, designed to bring the waters of the Sacramento River in northern California southerly to the Delta and Suisun Bay region near San Francisco, and to parts of the San Joaquin Valley, for irrigation and industrial purposes, is now well launched. Shasta Dam to form Kennett Reservoir is under construction and several miles of the Contra Costa Canal to supply the Suisun Bay region has been completed. Recent reports compared with those

Note.—Written discussion of this report will be transmitted directly to the Chairman for the information of the Committee.

[1] Presented at the meeting of the Sanitary Engineering Division, New York, N. Y., January 19, 1939.

[2] Engineering News-Record, November 24, 1938.

of 1937 indicate that the Shasta Dam has been increased in height from 420 to 560 ft, and the reservoir from 3 000 000 to 4 500 000 acre-ft (about 1 500 billion gal, or more than twelve times as big as Ashokan Reservoir).[3] The dam is rated as the second largest in the world. It will be 3 500 ft long and will contain 5 500 000 cu yd of concrete. The contract price was reported as about $36 000 000. Time allotted for completion of the project is 2 000 calendar days.

*Metropolitan Water District, Massachusetts.*—This project for taking water from the Ware and Swift rivers, 50 to 60 miles west of Boston, Mass., has been under construction since 1936, and the intake from the Ware River has been in use for several years. The hydraulic fill in the main dam for the Quabbin Reservoir on the Swift River was completed in November, 1938, and storage will begin in 1939. This reservoir has a capacity of 415 billion gal or six times that of the Wachusett Reservoir located about 35 miles from Boston and previously the main supply.

The supply for Boston and vicinity as a metropolitan district was begun in 1896 by extending the Boston supply westerly through construction of Wachusett Reservoir and Wachusett Aqueduct, connecting that reservoir with the previously most westerly Sudbury supply of Boston. Immediately following, the Western Aqueduct was built to form another carrier from the Sudbury system to the metropolitan district.

The work begun in 1926 has extended the supply to the Ware and Swift rivers, all in its effect substantially as planned in 1895. There has now been added, as a project of the U. S. Public Works Administration (WPA), a new pressure aqueduct 23 miles long to extend from the terminal chamber of the Wachusett Aqueduct into the district, eliminating much pumping and forming a part of an ultimate system which, largely in pressure tunnel, will loop around the district and eliminate most of the pumping. The immediate pressure aqueduct construction will consist of three miles of 14-ft pressure tunnel forming a by-pass under Sudbury Reservoir, a tunnel running parallel to, and superseding, Sudbury and Cochituate aqueducts, and 15 miles of reinforced-concrete, steel-cylinder pipe of various sizes up to 12.5 ft in diameter.

In the same grant is included some miscellaneous work embracing the clearing of Quabbin Reservoir, constructing shallow-flowage regulating dams on the Swift River and additional siphon pipes on the Weston Aqueduct to make the siphons equal in capacity to the remainder of the aqueduct. The Weston will become a stand-by or emergency aqueduct.

The work under the P.W.A. grant is estimated to cost about $16 000 000, and is scheduled for completion by June 30, 1940.

*Supply for Metropolitan New Jersey.*—A joint report has been made by officials of interested departments on the cost and feasibility of supplementing the present water supply to Metropolitan New Jersey by water to be taken from the Delaware River a few miles north of Trenton, N. J., down the abandoned bed of the Delaware and Raritan Canal, thence being pumped to a reservoir at Elevation 450 and distributed to various of the cities. An average draft of about 150 mgd with peak up to 230 mgd was estimated for this study.

[3] *Proceedings*, Am. Soc. C. E., March, 1938, p. 509.

The canal formerly had a capacity of 400 mgd, but gagings showed its present capacity to be about 100 mgd owing to encroachments and filling in of the canal prism.

The study indicated that a pipe in the canal bed, rather than the open channel of the repaired canal, would be the more effective, as well as the more sanitary, solution. This project like the new Delaware project of New York City has an interstate nature, and the validity of the old canal rights would be a point of inquiry.

*Delaware River Supply for New York City, N. Y.*—The construction of the new supply of 540 mgd, including 100 mgd from Rondout Creek not in the Delaware water-shed, is proceeding with the customary modern speed of shaft sinking and tunnel driving. The first stage of the Delaware system now under construction will comprise two impounding reservoirs—the Neversink and the Rondout—six miles of tunnel connecting these reservoirs and an 85-mile aqueduct to the existing Hill View distributing reservoir at the northerly boundary of the city, all at an estimated cost of about $210 000 000.

The 85-mile aqueduct will be a continuous, deep pressure tunnel from 13.5 to 19.5 ft finished diameter inside the concrete lining, and at depths below the general ground surface of from about 300 to about 1 000 ft. This tunnel will be driven from thirty-one shafts from 310 ft to 1 350 ft deep.

To date twenty-nine of the thirty-one shafts have been completed and lined with concrete, and tunnel excavation has been started in both directions from the bottoms of these shafts; one of the other two shafts is nearing completion. More than 17 000 ft of shaft and two miles of tunnel had been excavated at the end of the year.

Exploratory work is proceeding at Neversink dam site and water control works are under construction at the site of the dam for Rondout Reservoir.

Contracts have been awarded to the value of 50% of the cost of the entire first stage of the Delaware project and it is expected that contracts for the balance of the work will be advertised at short intervals. The contract times for completions, if not exceeded, would permit the first water to be supplied to the city sometime in 1944, additional aid from storage in the existing 30 billion gal Kensico reservoir to be received about a year earlier.

*South Side Filter Plant for Chicago, Ill., Supply.*—This project, the first filter plant for the water supply of the City of Chicago, has recently been begun just north of 79th Street. A breakwater to diminish the force of waves against the structure, and the somewhat U-shaped coffer-dam to enclose the site are under construction. The site is now mostly under water up to 14 ft deep in the bottom of Lake Michigan, since it was found more economical to make land than to buy enough existing land.

The plant will take its water through the near-by existing intake tunnel with intake about two miles off shore. A branch pipe from this tunnel will lead to the filter plant which will consist essentially of a low-lift pumping station, two-storied mixing basins of 40-min capacity with horizontal flocculators, two-storied settling basins of 3-hr capacity, eighty filters, each of 5-mgd capacity at a rate of 2.5 gal per min per sq ft, and a clear water reservoir of 20 million gal.

The cost is estimated at $21 000 000. The plant will supply filtered water to the Southern District of Chicago and thirteen south side suburbs, the total population served being about 1 700 000.

*Bartlett Dam, Highest Multiple Arch Dam.*—The Bartlett Dam, on the Verde River about 54 miles north of Phoenix, Ariz., is scheduled for completion in May, 1939. It is 287 ft high and impounds 201 500 acre-ft (about 65 billion gal). The water is to be used for irrigation of land in Salt River Valley and Salt River Indian Reservation.

*Hartford, Conn., Metropolitan Supply.*—Saville Dam (named for Caleb Mills Saville, M. Am. Soc. C. E., its Chief Engineer and the Manager of the District), largest dam in Connecticut, is nearing completion. It is a rolled-earth structure with concrete core, height 137 ft, spillway 200 ft long, producing a supply of 54 mgd. The cost of the dam will be about $2 850 000.

*Water Supply for Westchester County, New York.*—This county now has many separate supplies, several of which take water separately through pipes connected with one or another of the aqueducts serving New York City but passing through Westchester County. Two plans have been reported on for a consolidated Westchester County supply, one being for a new supply from Conocus Creek near Peekskill, N. Y., at an estimated cost of about $22 500 000, and the other for a consolidated supply from New York City aqueducts estimated at $9 900 000. Filters are included. The usual objection to the first plan has been raised by Putnam County in which the impounding reservoir for the plan would be located.

*Grand Rapids, Mich.*—A new supply for Grand Rapids, Mich., to be taken from Lake Michigan, will be begun with Federal (P.W.A.) aid during 1939. The present supply is from the Grand River. The new supply will include an intake and intake pipe line about 8 000 ft long, a pumping station at the lake shore, about 30 miles of pipe not less than 42 in. in diameter, possibly an intermediate booster station along the pipe line and covered reservoirs already in part under construction and of total capacity of 40 million gal. About five miles of the pipe line may be in tunnel to reduce the pumping head. The water will be filtered in the existing plant. The present daily use averages about 18 mgd, with a maximum peak day of 40 million gal.

*Additional Water Supply for Salt Lake City, Utah.*—Agreements have been reached by which the U. S. Reclamation Service will construct, for joint use of the Metropolitan Water District of Salt Lake City and vicinity and the Provo River Water Users Association, a reservoir of 150 000 acre-ft (50 billion gal) capacity at Deer Creek on the Provo River water-shed. A 66-in. concrete conduit 40 miles long will conduct the water to Salt Lake City and vicinity. As all the water of Provo River was long ago appropriated, the water will come by an existing diversion canal, enlarged for the purpose, from Weber River and Duchesne River, the latter a tributary of the Colorado River.

*Muskingum Dams in Ohio.*—Fourteen dams giving protection from floods for an area in Ohio 100 miles wide and 140 miles long have been completed. The population of the district is 675 000. The main reservoirs are on the three principal tributaries of the Muskingum River—the Walhonding River, the Tuscarawas River, and Wills Creek. With a total storage of 1 559 000

acre-ft the dams will reduce the flood crest of the Muskingum at Zanesville, Ohio, by 15 ft.

## Floods and Other Accidents to Water Supplies

*Floods in Southern California, March, 1938.*—The Southern California coastal plain was swept by a flood from March 2 to March 5. A heavy rain storm covered the full width of California, but was especially severe in the steep mountain area bordering the Los Angeles-Riverside plain on the north. The death loss was approximately 200, and the property loss was estimated at over $50 000 000. The total rain in the coastal plain and foothills varied from about 5 to 13 in. for the entire storm. It rose to about 11 in. in 8 hr and 15 in. in 24 hr in the mountain area. The damage was mainly to bridges, railroads, and highways, although some of the distribution water mains were exposed.

*Flood Damage on Eastern Slope of Rocky Mountains.*—A rain storm of three days' duration at high intensity caused damage by floods on September 3, 1938, estimated at one-quarter million dollars over a latitude from the south boundary of Wyoming to Colorado Springs, Colo., a distance of 150 miles. The recently constructed flood-regulating Sullavan Dam, 25 miles above Denver, Colo., and having an automatic flood control regulator, passed a successful first test in this storm.

*The New England Hurricane and Floods of September 21, 1938.*—The hurricane that struck New England and the eastern part of New York on September 21, 1938, apparently exceeded all previous storms of which there is any knowledge as having occurred in this general area, both as to velocity of wind and resultant damage. In addition, the damage was much increased by the practically simultaneous arrival of river floods accumulated by extraordinary rains, giving a total of about 16 in. of precipitation in the nine days preceding the hurricane, and causing run-offs reaching more than double the previous high records in some observed cases (the Ware River for example).

The usual tropical storms of the hurricane season, which develop in the West Indies section, follow, roughly, the United States coast line to Cape Hatteras and then veer to the east over the ocean. This hurricane, however, found opposing higher pressures to the east and also the west, and hence took an unusual course, almost due north, passing over Long Island and through the central part of New England where it joined its havoc to that of the river floods.

The lowest recorded pressure was 28.04 in. at Hartford at 4:17 P.M. on September 21. The maximum wind velocity recorded was 121 miles per hr at Blue Hills, near Boston, with 87 miles per hr at Providence, R. I., 82 miles per hr at Block Island, R. I., 73 miles per hr at Boston, and 70 miles per hr at New York City. The wind, aided probably by very low barometric pressure near the center of the storm, piled the water up on the south shore of Long Island and the north shore of Long Island Sound, causing many millions in property damage and great loss of life due to the rise of the salt-water level, the tremendous force of the waves acting from these higher levels and the wind destruction. At Providence the water rose 13 ft 9 in. above mean

high tide, whereas the previous sea-water level at Providence was recorded on September 23, 1815, when the water rose 11 ft $9\frac{1}{4}$ in. above mean high tide. The rises were very sudden and not coincident with high tide.

The only two storms that apparently approached the one of last September in intensity and rise of tide were those of August, 1635, when it was stated that the tide rose 20 ft in Boston and came in twice in a 12-hr period. On September 23, 1815, a hurricane that appeared to be very similar to the one of 1938, visited New England, although it is reported that in New York this hurricane only showed moderate wind; and it is stated that the storm did not extend far out to sea. None of the other gales and hurricanes appear to have had a wind velocity exceeding about 60 miles per hr.

In Rhode Island the damages to water supply systems were tabulated by James J. Dillon, Sanitary Engineer and Chemist, Rhode Island Department of Public Health, as: (1) Power failures; (2) leaks on distribution systems; (3) destruction of reservoirs; (4) pollution of reservoirs by salt water; (5) flooding of purification plants; (6) decreases in filter runs; and (7) increase in chlorine demand. He stated that all of the forty-one public water supplies in Rhode Island were confronted with some sort of problem which had to be solved to maintain service. Practically all outside electric power service was cut off, hydrants and mains were broken by fallen trees and buildings that collapsed; one dam was washed out, one water supply reservoir was flooded by the salt water going over the top of the dam, and one purification plant was flooded, thus delivering contaminated water into the mains, requiring almost a week for their sterilization.

In Massachusetts, the damage to water works systems was estimated at about a half a million dollars. Boiling orders were issued in connection with the water supplies delivered to thirty-three communities, and in fourteen cases the sources of supply were flooded, and it was necessary to rely on storage distribution reservoirs until the flood had receded. There were ten cases of power failures at pumping stations or treatment plants, and in two municipalities the standpipes were damaged.

In eastern New York, the polluted water entered several of the public water supply systems, and chlorination treatment was interrupted due to power failures. On Long Island the electric power failure was complete in Suffolk County and mains were washed out at four communities by the breaching at many places of the outer bars and dunes behind which were laid the mains which served summer homes. The damage to mains was relatively small but hundreds of houses were demolished, and unless, and until, they are rebuilt the mains will be of little use even if repaired. There were floods in the Adirondacks and some of the reservoirs were completely filled and clogged by the deposits of gravel and débris which, together with breaks in water mains, left the consumers without water.

In Connecticut the results were not so serious, there being a small dam on the South Glastonbury Water Company system, a small dam on the East Hartford system, and a dam on the Natchaug River (where Willimantic gets its water) that were washed out. Power failures interfered with the operation of pumping stations, filter plants, and, in some few cases, chlorination appa-

ratus; but the State Department of Health has discouraged the use of chlorinating apparatus dependent upon electric motors for the operation of pumps and injectors, and this hurricane furnished another lesson in this regard. Boiling notices were issued wherever there was any question as to the purity of the supply.

Damage to timber was very great, in New Hampshire particularly.

*Frequency of Large Floods.*—Attention is directed to the very heavy floods which occurred in New England in 1927, again in 1936, and again with the hurricane in 1938. By the theory of probability floods of such magnitude are not expected to occur more often than about once in fifty or one hundred years. The grouping of three major floods in an 11-yr period again emphasizes the danger which engineers sometimes encounter that non-technical men in authority will think of a 100-yr flood as one which will not occur for another hundred years and therefore need not be guarded against by the present generation.

*Slide at Fort Peck Dam.*—On September 21, 1938, about 8 000 000 cu yd of fill in the Fort Peck Dam slid up stream, burying eight men and carrying down twenty-six others who, however, escaped with their lives. The slide extended across the core pool and included also part of the down-stream core-retaining embankment. Results of the investigation of cause have not yet been published. A failure of the foundation shale was suggested as a possible cause. This failure is apparently of the same sort on a larger scale as occurred at Necaxa and at Calaveras dams, although the Fort Peck Dam has more conservative slopes than these earlier and bolder undertakings. An account of the failure with comparisons of grain sizes of core material and records of consolidation of the core will be awaited with interest.

### State Versus Federal Jurisdiction over Water-Sheds

*Flood Control in New England.*—The several excessive floods in New England in recent years have caused Federal and State study and planning, but action has been delayed as reported in the 1937 report[3] by controversy between State and Federal officials as to the ownership of land to be purchased for reservoir and other control works and as to the Federal Government's insistence on ownership and control of future hydro-electric power possibilities to which the States have refused to agree. The Federal Government unexpectedly refused ratification of pacts satisfactory to the affected States, principally Connecticut, Massachusetts, and Vermont, and instead passed an act permitting the Federal Government to acquire the land and act independently of the States. The New England States, beginning with Vermont, have agreed to resist legally, and the President has apparently let it be known that the first proposed construction, that in Vermont, will be abandoned. The controversy is apparently now going to the floor of Congress in an attempt of United States Congressmen to have repealed the act to permit Federal action without State ratification.

### Tendencies in Financing Water Works Improvement

In the Middle West there has been noted by the Committee a pronounced trend in water works toward revenue bond financing of new construction.

This doubtless arises partly from the influence of the Reconstruction Finance Corporation (RFC) originally, and later the P.W.A.

By revenue bonds is meant an obligation which, both as to interest and principal, depends solely upon the earnings of the utility and which expressly creates no obligation upon the municipality, and which is therefore entirely outside of the debt limitations fixed by statute. In most cases the revenue bond laws require the municipality to maintain rates for service sufficient to pay the operating costs, maintain the property, pay the bond interest and amortize the debt as it matures. In one State a law that did not require payment of operating expenses out of revenue was defeated in 1938.

Prior to 1929, water revenue bonds demanded a higher coupon rate than general obligation bonds. It was found during the depression, however, that cities defaulted on their obligations quite generally, but revenue bonds supported by the revenues of the water utility "came through" in excellent shape, so that, at the present time, the coupon rates of the two issues are quite comparable.

The low interest rate for water revenue bond financing has resulted in the transfer of many privately-owned water works to municipal control. In general, the bond rates have been approximately 3 per cent. Utica, N. Y., issued $7 900 000 at a cost of 2.63 per cent.

Coincident with a considerable amount of water works revenue bond financing, there have been many illustrations of sewer bond financing, and due to the close relationship between water and sewage a considerable number of instances of combined water and sewerage issues. It is believed that there is a closer co-ordination of water and sewer department organizations in municipal operations than formerly. In general, water revenues and sewer revenues are kept entirely separate, although in large part they are both based on water use.

### Effect of W.P.A. on Construction

It has become increasingly difficult to secure figures from contractors specializing in the construction of small distribution mains. Inquiry develops that in the City of Chicago, where formerly there were dozens of sewer contractors, the city, although engaged in laying many miles of sewers annually, has not let a single sewer contract since 1931. All work has been under the W.P.A. Similar tendencies have been noticed in the East also.

The W.P.A. has also exerted a minor influence on the type of water works building construction. At Lansing, Mich., a 20 mgd softening plant is being built wholly of reinforced concrete as compared to the usual brick or stone superstructure. The principal reason was the desire to spend as large a proportion of the total for labor and as small a proportion for materials as practicable.

### Trends in Construction of Earth Dams

The building of large earth dams continues—both by the rolled-fill and hydraulic methods—with design and control of construction largely influenced by the relatively recent developments in the science of soil mechanics.

The increasing size and speed of rolled-fill equipment have reduced resulting unit costs, but the hydraulic method proves economic in some cases. The

Sardis Dam in Mississippi (12 million cu yd) and the Keystone Dam in Nebraska (25 million cu yd)—both started in 1938—are full hydraulic fill construction using dredges of great capacity.

Although more rationalized design has been made possible by the developments of soil mechanics, experience is indicating the need of caution in applying laboratory results.

*Rolled Fill Dams.*—In relation to rolled-fill dams it is becoming evident that:

(*a*) Almost any type of soil can be used, if proper drainage is provided;

(*b*) Drainage of the down-stream portion of dams consisting of relatively impervious material is important and possible by the placing of pervious sections or blankets—with adequate provision of graded transition filters;

(*c*) Compaction of plastic material should be limited to that degree which can be maintained, after saturation, by the weight of overlying material, and over-compaction is possible and undesirable;

(*d*) The most careful investigation of foundation soils is essential, and horizontal shears under an embankment—which may be increased by over-compaction of fill—may be a controlling factor in design;

(*e*) The value of concrete keywalls in joining earth fills to rock is questionable, and such walls have been omitted in many designs;

(*f*) Sand sections which will be saturated when the reservoir is full must be compacted below the "critical porosity";

(*g*) Excess sprinkling of sandy gravel material and the resulting water sorting and gradation of particles reduces the amount of necessary rolling;

(*h*) For granular material a heavy tractor is a highly efficient compacting implement, and rolling of such material increases compaction by only a small amount; and,

(*i*) There is a tendency to flatter slopes, and, in general, somewhat greater factors of safety.

*Hydraulic Fill Practice.*—In hydralic-fill practice:

(1) There is a tendency toward much narrower cores, and core side slopes of one vertical to one-third horizontal are becoming quite common;

(2) Experiments are being made to determine the best method of eliminating sand tongues in the cores by towing a scarifier along the bottom of core pool, by using sand jets to mix the sand with the underlying fines, and by excavation, by small dredge pump on barge, of a trench on each side of core pool and thus refilling sand tongues with adjacent fine material;

(3) An interesting method of constructing hydraulic fill dams is the long-distance conveying of material from borrow-pits to hog box by rubber belts. At the Quabbin Dam of the Metropolitan District Water Supply Commission (Massachusetts), belts, 36 in. to 42 in. wide, carry the material a distance of three-quarters of a mile—with end section at borrow-pit (800 ft long), arranged to follow the shovels by swinging in semi-circle.

(4) Rubber-lined steel pipe—used at the Quabbin Dam for two construction seasons—has been a success in the straight runs; but in bends and

adjacent sections the adherence of the lining is destroyed by the turbulence and tearing effect of the sluiced material;

(5) The vibration of the shells of hydraulic fill dams by the use of caterpillar tractors has been recently required by the Chief of Engineers, U. S. Army.

(6) A novel procedure in the construction of the Keystone Dam is the pumping of the shell material from the river bottom, which contains little or no fines, and the obtaining of the fines from the loess deposit in the adjacent hills, pumping directly into the core pool; and,

(7) In both rolled-fill and hydraulic dams, settlement gages—of which the Subcommittee on Consolidation of Embankment and Foundation Materials of the Soil Mechanics Division has developed several types—are being installed generally. At the Quabbin Dam steel wells 3 ft in diameter, with ports, provide opportunity for sampling and tests of condition and consolidation of core material.

### Improved Portland Cement

In the 1937 report[3] there was given a description of cement, in use by the Board of Water Supply of New York City, which was manufactured with more careful control of composition, burning of the clinker, and the handling of the clinker. Upper limits are set to percentages of iron, alumina, gypsum, magnesia, and the alkalies, and lower limits to both iron and silica. A minimum of water is used in cooling the clinker which must be stored under cover, and ground dry into cement within six weeks of its burning. A sugar test is used for free lime content, lime in excess being an indication of under-burning. This cement continues to be used in the new Delaware supply works of New York City. It was also used in the Fort Peck Dam and is being manufactured on the Pacific Coast. The production of cement which will make durable hydraulic structures is of the utmost importance to water works engineers, and if this cement lives up to its promise its introduction will ultimately be regarded as the beginning of a new and better epoch in construction.

### Centrifugal Pump Developments

There have been no outstanding developments in centrifugal pumps during 1938, but certain trends have been accentuated.

In deep-well turbine pumps there is an increased tendency to use water-lubricated units, particularly among operators of textile, chemical, and similar plants who feel that even a drop of oil in the water may do some damage to the product. The development of cutless-type rubber shaft bearings and the non-corrosive sleeves placed over shafts at bearings by some manufacturers have contributed greatly to the successful development of this type of pump. Very satisfactory operation is being obtained even with deep settings and high speeds.

In the ordinary type of centrifugal pump designs have remained substantially unaltered, but there has been a slight increase in efficiencies guaranteed and high pressures attained, particularly in boiler-feed pumps, of which there are now many units in operation successfully handling pressures of 1 600 lb per sq in. or more at relatively high temperatures.

Steam turbine driven centrifugal pumping units have been built in increased numbers. Large units have been supplied for Atlanta, Ga., Chattanooga,

Tenn., Chicago, and other places; and this is noteworthy because the trend for the past few years has been toward electric-motor driven units, chiefly because of their lower first cost, simplicity of installation, and operation.

The steam-turbine unit meets the demand better than does the electric unit for continued high efficiency with the varying rates of pumping required in most systems. This is because the efficiency of both steam turbine and pump remain practically unchanged with rotative speeds within a range 10% either side of normal. The speed control is simple, moreover, involving no complicated electrical apparatus.

Combination or split pumping units of the kind described below are becoming more common. Each unit consists of a steam turbine: (1) Driving, through speed reduction gearing, a pump (2) of relatively high head and horsepower located on the main floor, and an electric generator (3). A second pump (4) of same capacity as, but much lower head than, the first pump is electric driven with power from the generator (3). It is generally located in a pit to take suction from a low-water level and it discharges into the first pump with which it is in series. Speed and capacity regulation of both pumps is effected by changing the speed of the steam turbine which affects the speed of the electric-driven pump by change in number of cycles of the current generated. The economy of direct driving from the prime mover is obtained with the pump requiring the greater part of the horse-power. The electric generator may be proportioned to furnish power for all station auxiliaries. Transformers, elaborate switch-boards, and controllers are eliminated. The economy of this split unit, even including the very considerable fixed charges for boiler plant and turbine, is generally considerably better than that of electric units with purchased power.

### Progress in Design and Construction of Pipe Lines

*Reinforced Concrete Pipe.*—There is an increasing use of rubber rings in place of jute-filled lead gaskets for joints of reinforced concrete pipe. These rubber-ring gaskets, about $\frac{5}{8}$ in. in diameter, are soaped and set in a groove in the steel collar at the spigot end. The steel collar at the bell end slides closely over this rubber, enclosing it and compressing it into a space surrounded closely by the steel. Mortar in the clearances outside and inside the pipe complete the closure. The laying costs are considerably reduced as all calking is eliminated. The volume of rubber in each gasket is checked separately by displacement in water.

*Steel Pipe.*—Welded pipe continues to be the almost invariable type of steel pipe in use. Shop joints are of the butt type of nearly 100% efficiency, giving an interior smoothness of great advantage in carrying capacity as compared with riveted pipes formerly used. Field joints are generally of one of three kinds: (1) Butt welded, (2) with bells about 2 in. long formed at one end of each 30-ft or 40-ft section, into which bell a plain end fits and is fastened by welds inside and outside; and (3) plain ends with Dresser couplings. In a comparatively few pipe lines, including those in New York City, field joints are made with longer formed bells and riveted connection. Some companies make a general practice of cleaning off all mill scale with steel shot.

*Small Steel Pipe in District Systems.*—There have continued to be some installations of steel pipe in the smaller sizes, about 4 to 16 in. in diameter, with thicknesses of less than $\frac{1}{4}$-in., protected with bituminous coatings.

*Cement-Asbestos Pipe.*—Cement-lined pipe with sleeves and round rubber gaskets continues to be used in considerable quantities. Another type of cement-asbestos pipe, built up by spiral-wound ribbons of cement-asbestos instead of by a single ribbon of width equal to the length of the pipe and with Dresser type joints, has been introduced into the United States and is being made in small quantities during completion of the development of methods of manufacture.

*Cast-Iron Pipe.*—Cast-iron pipe continues to be improved in quality. There is a growing practice for important jobs in the use of the new formulas for design developed by Sectional Committee A-21 (American Water Works Association) on specifications for cast-iron pipe, taking account of earth loads as well as internal pressures.

The bulk of cast-iron pipe in sizes up to about 20-in. is now made centrifugally, or in horizontal green sand molds with multiple-gate pouring; but pit-cast pipe is said now to be "holding its own" in proportion of total pipe manufactured. Sand-spun pipe up to 36-in. in diameter is being made and long lines of 30-in. sand-spun pipe have been laid. Larger spun pipes up to 42-in. and more are in contemplation.

*Cement Joints for Cast-Iron Pipe.*—Cement joints for cast-iron water pipe have been used rather generally on the Pacific Coast for many years, and sporadically elsewhere in the country. There appears to be a revived study of the use of cement joints, and one pipe manufacturer has made, and is making, extensive tests which so far have shown excellent results as to leakage both in undisturbed joints and in joints deliberately jacked out of line.

It is reported that a 30-in. cast-iron main about 15 miles long for the City of Corpus Christi, Tex., was laid with cement joints, with excellent results as to water tightness. The City of Los Angeles has laid many cast-iron lines, some of good size, with cement joints.

Cement joints are known to be satisfactory for small as well as large sizes of pipe in the equable climate of California, and have been used a great deal for low-pressure gas systems in the East. They have caused many pull-apart breaks in small gas pipe in the colder States. Such pipes are laid at relatively shallow depths. For the larger pipes the pulling-apart danger does not exist.

*Loading Cast-Iron Pipe for Shipment.*—A few breakages in transit occurred in the first shipment, by fast trans-continental freight, of 30-in. cast-iron pipe for the 7.5 mile pipe line at Corpus Christi. In a special investigation by the pipe manufacturer, using deflection measuring instruments, it was found that these pipes were constantly flattening and opening $\frac{1}{2}$ in. or so either side of the round shape, even when passing usual track unevennesses and much more when a fast-traveling train was stopped quickly by air breaks. The action during sudden stops is explained as one of suddenly-arrested momentum of the top of the pipes, which lie longitudinally of the car, causing downward deflection at the forward end and upward deflection at the rear end of the pipes. All

breakages were stopped when bands of steel were cinched tightly around groups of pipe in the cars. This remedy was adopted as general practice.

Your Committee would be glad to hail the adaptation of magnetic methods to the discovery of cracks in cast-iron pipe as they hang over the trench before being laid, as they believe that most unexplained failures—and that means most failures—are due to incipient cracks formed at the foundry or during subsequent handling and transportation.

*Welded Branches in Cast-Iron Pipe.*—At least one foundry is now furnishing, on request, branches formed by welding branch outlets with bronze on to full-length pipe, instead of cast as special fittings.

*Steel Non-Pull-Out Dogs in Mechanical Joints of Cast-Iron Pipe.*—Hardened steel dogs about $\frac{1}{2}$ in. wide by the thickness of the joint are being furnished by at least one manufacturer to prevent end pull-out of mechanical rubber-gasket joints in locations, such as at bends, where such preventives are necessary. The bolts which are used to pull in the gland at the same time squeeze the steel dogs between gland and gasket so that the sharp edge of each dog digs into the spigot of the pipe. The number of dogs is determined to correspond with the estimated end pull.

*Design of Fittings.*—In connection with the work of the Sectional Committee A-21 on Cast Iron Pipe and Fittings, one manufacturer is investigating improved design of large fittings—to test which, on a full scale, is very expensive—by the use of $\frac{1}{4}$-scale models in cast iron. The use of hard rubber models is also contemplated to permit easier study of the stresses in such complicated fittings as tees, crosses, and wyes.

*Interior Protection of Pipe Lines.*—Coal-tar dip yields very slowly to more permanent cement mortar or bituminous enamel linings owing apparently to the persistent ignorance of water superintendents of the great effect of small roughnesses on the carrying capacity of pipe lines. The Committee's canvas a year ago showed percentages of cement-lined pipe furnished by various foundries varying from zero to a maximum of 13%, except for one New England company where the percentage was about 50. A recent canvas by a traveling demonstration truck of one of the foundries showed much interest in improved linings in some States west of the Mississippi River, but little interest in the Lake States where the water is harder; yet your Committee has personal knowledge of many pipe lines even in this lake country with coefficients well below 100, and even 60 in smaller pipes.

Cast-iron pipe, having a spun cement lining $\frac{1}{16}$ in. to $\frac{1}{8}$ in. thick according to diameter, and coated with asphalt or not as desired, is now offered by at least one foundry at no cost above tar-dipped pipe, and all cast-iron pipe manufacturers are licensed to use the process.

Spun bituminous (tar) enamel is becoming standard for steel pipe and is used some for cast-iron pipe.

*High-Alumina Cement Lining.*—One cast-iron pipe manufacturer has reported results so far perfect in lining mine drainage pump discharge pipes with a spun lining made of high alumina cement. The lining is said to be smooth, hard, and glassy, without cracks, and apparently unaffected by this very corrosive water.

*Spun Cement Lining for Steel Pipes.*—In the first feeder main from the main aqueduct of the Metropolitan District of Southern California there were installed in 1938 over seventeen miles of 55-in. and 51-in. welded-steel pipe, $\frac{3}{8}$ in. thick, lined with cement mortar about $\frac{1}{2}$ in. thick spun in, and an exterior protection of reinforced gunite about $\frac{3}{4}$ in. thick with tar enamel beneath the gunite in very corrosive ground.

The East Bay Municipal District has installed, during the past year, 25-in. welded-steel pipe $\frac{3}{16}$ in. thick and 36-in. pipe $\frac{1}{4}$ in. thick with spun cement linings approximately $\frac{5}{8}$ in. thick and reinforced gunite exterior covering about $\frac{3}{4}$ in. thick. Other water supplies on the Pacific Coast have also installed spiral-welded and straight-seam-welded pipe of 15-in. to 21-in. diameter with steel of 10 or 12 gage, spun cement mortar lining $\frac{1}{2}$ in. thick and double wrap of asphalt and asbestos felt.

*Durability of Interior Protection for Pipe Lines.*—Long-time tests of improved linings are few, and inspections and flow tests are not made as they should be to furnish reliable data for specifying new work. A few crumbs of information are picked up from time to time but nothing systematic. The brushed tar enamel coating, which was the first tar enamel applied in this country (namely, on a 66-in. steel pipe in Brooklyn, N. Y., under specifications of one member of the committee) was last inspected in 1932 after seventeen years of use and had a few tubercles, mostly along the lock bar, and a considerable number of water blisters. Otherwise it was in excellent condition and immeasurably better than the asphaltic coating formerly applied. This coating was only $\frac{1}{32}$ in. thick. Later tar enamel coatings have been $\frac{1}{16}$ in. or more thick, and recent ones have been spun in.

Inspection of these thickly brushed coats at age ten years or more have shown that they are still nearly perfect in New York Catskill water, which is very corrosive. Later New York City tar enamel coatings have been spun. These have not been inspected, and there are no reports of inspection or flow coefficients of tar enamel coatings elsewhere after some years of service.

Cement linings, both spun into cast-iron pipe and cast into reinforced concrete pipe, have been tested or inspected (or both) in a number of cases after ten years or more, and without exception found in good condition with little reduction in carrying capacity. Cut-outs made in 1938 from a 12-in. cement-lined cast-iron pipe in Charleston, S. C., laid in 1926 (age twelve years) show some blackening but practically no corrosion of the lining. This pipe was in the plant of J. E. Gibson, M. Am. Soc. C. E., who inaugurated the cement mortar lining for cast-iron pipes in about 1920 and believes it to be durable. A cement plaster coat over a piece of steel pipe in the New York Catskill system showed marked chalking and loss of strength to a depth of about $\frac{1}{10}$ in. after eighteen years.

Masonry aqueducts in soft water frequently show sanding of the concrete surface, and in very soft water a few aqueducts and concrete pipes have been badly damaged.

Old cement-lined stove pipe in New England and in Virginia, lined with natural cement, and either with or without tar over the steel under the cement, have in many cases lasted a long time, and where they have failed have done

so generally from modern pressure too high for their thickness and hand riveting, from their poorly made sleeve cement joints, or from rusting where outside protection was missing. Natural cement is a different product from the Portland cement now used, however.

Your Committee thinks there is need of active study and investigation of the effect of time on various kinds of coatings through examinations and flow tests, and a collateral study of effect of the nature of the cement on the durability of linings, lest carelessness in selection of material and perhaps too great economy in thickness defeat the promise now offered by these greatly improved protective processes available for water supply.

Asphalt coating over cement linings are becoming more common, especially where the water is soft and the lining otherwise causes early causticity. The question is being raised whether it is not always wise to take advantage of whatever additional life of lining is afforded by this cheap protection. Measurements of coefficients of cement-lined pipe with and without the asphalt protection are needed on new pipe and again from time to time as it ages.

*Effect of Treatment of Water on Flow Coefficients.*—Such additional evidence as has developed along this line confirms the statement in last year's report[3] that tuberculation is not prevented by treating the water to put it in chemical balance.

*Exterior Protection for Pipe Lines.*—For the exterior of steel pipe bituminous tar enamel, with or without asbestos felt wrapping, is generally used for protection. In markedly corrosive ground gunite or a mortar wrapping bound on by cotton fabric is sometimes used. Similar wrapped mortar coverings in thicknesses of $\frac{7}{8}$ in. or more are now being used to a limited extent for cast-iron pipe, notably in Charleston, S. C. This wrapping is done at the foundry and is reasonable in price.

The cathodic method of protecting pipe from exterior corrosion is being more often looked to and planned for as the ultimate protection for long gas and oil lines to be used if and when the coatings fail; water supply engineers are becoming conscious of this possibility, at least for steel pipe, and there is some thought being given to it for cast-iron lines. Technique has not been developed for cast-iron lines, however, and it is not known to what extent, if at all, various kinds of joints will lend themselves to this type of protection.

*Water Hammer.*—The principles of the occurrence of water hammer in force mains with shut off of power are becoming sufficiently understood to be generally recognized, and the water hammer provided for in more important pipe lines. The simplest and smallest effective means of prevention in such cases appears to be to check the tendency at its inception by the use of quick-opening relief valves actuated from the initial drop in pressure which immediately follows the power shut off. A special check valve with final closure retarded in a predetermined and governed manner is a part of this plan. In ordinary distribution system work, little progress has been made in general knowledge of water hammer, and we are still using old water-hammer figures more or less "picked out of the air" in the design of pipes. The subject merits investigation by water supply operators and engineers. A reliable and reasonably-priced gage that may be generally used for correctly recording

water hammers is the first requisite for the accumulation of the needed data. A committee might well be charged with this matter.

### Progress in Knowledge of Utilization of Ground Water Supplies

Study of recession of ground water and encroachment of salt by overpumping is proceeding at many points. Those of Pecos Valley, in Long Island, N. Y., and New Jersey were noted in the Committee's report last year.[3] Information regarding similar studies near the Gulf of Mexico have been prominently reported during the past year. These studies have been made in connection with a study of water supply for the City of Houston, Tex., and more academically by the U. S. Geological Survey, in co-operation with the Texas State Board of Water Engineers. Eighty million gallons daily are being withdrawn from underground sources at Houston, and the pumping level which was formerly some 50 ft below the surface of the ground is now depressed to as much as 220 ft. It seems that here, as in many other places, water works operators have been engaged in mining the accumulated water treasure of the earth and not allowing it to be replenished.

### Progress in Treatment of Water

*Great Increase in Number of Treatment Plants.*—Owing to Federal stimulation and aid in financing, an unusual number of water purification plants have been built or are projected. Among the larger plants may be mentioned those in Chicago, Ill.; Evansville, Ind.; Atlanta, Ga.; St. Paul, Minn.; Minneapolis, Minn.; Providence, R. I.; and Corpus Christi, Tex. New plants or extensions have been constructed, are under construction, or are in the study stage, for Albermarle, N. C.; Caldwell, Ohio; Fredericksburg, Va.; Augusta, Ga.; Ypsilanti, Mich.; Lockport, Mass.; Lawrence, Mass.; Fredonia, N. Y.; Mandan, N. D.; Union Town, Pa.; Grand Junction, Colo.; and a large number of smaller cities. Softening plants have been recently completed, are under construction, or contemplated, at Shelby, Ohio; Monroeville, Ohio; Bellevue, Ohio; Flint, Mich.; Clarksburg, W. Va.; and for the Metropolitan District of Southern California.

*Improvements in Sedimentation.*—There is increased interest in a method developed by Mr. Chas. H. Spaulding at Springfield, Ill., a few years ago, of separating the suspended material from water in process of softening by chemical treatment and precipitation.

Minneapolis is using a special type of settler for the extensions to its purification plant which includes softening. This device in essence is a funnel-form tank in which the water enters at the bottom and flows upward. An agitator at the bottom keeps the sludge in suspension so that the incoming water will have to pass through a blanket of sludge. The vertical rise of water just below the overflow weir is about 2 in. per min.

At Minneapolis, for a capacity of 120 mgd, there are twelve clarifiers each with diameter of outer truncated cone at top 86 ft and at bottom 58 ft. The bottom of the inner cone has a diameter of 50 ft and the top 16 ft. Below the base of the inner cone is a cylindrical chamber 58 ft in diameter in which

extended arms will rotate to keep the sludge suspended in the water. Part of the sludge will be withdrawn from this chamber from time to time.

*Activated Carbon.*—Separate filters with granular activated carbon have been used in a few plants with unvarying success, and many small household or apartment size units are in use as was the case years ago with the less efficient bone char units. The initial cost of duplicating the filter units to provide for granular charcoal beds as well as sand beds and the improvement in efficiency of powdered activated carbon have combined to side-track the increased use of granular carbon filters. Tastes and odors from industrial wastes will sometimes get by the powdered carbon protection, however, and there is great need for more activity on the part of boards of health to compel abatement of these nuisances which are becoming more general in distribution as America is rapidly becoming a nation of chemical manufacture.

*Bentonite and Bleaching Clays.*—Study in the use of these natural purifying agents is being made for certain waters.

*New Inquiry into Effectiveness of Purification and Sterilization Processes.*—This inquiry, begun last year, following certain unexplained outbreaks of dysentery, has progressed, but it is too soon to expect definite results. However, more attention is being given generally to the efficiency of sterilization, and it seems probable that the water of the future will receive a higher chlorine dosage.

*Use of Silicates.*—Use of silicates as an aid to coagulation is being investigated at a few plants with encouraging results.

*Recovery of Lime at Softening Plants.*—Experiments have been made, in connection with the proposed 100 mgd plant for softening a part of the Colorado River supply for the Metropolitan Water District of Southern California, in the recovery of lime used in the softening process. This will require separation and disposal of the magnesia derived from the water, but will permit repeated re-use of the lime and will greatly simplify the usually very difficult job of disposing of the sludge.

## Miscellaneous New Designs and Ideas in Water Supply

*An Insulated Water Meter Connection.*—Complaints of cloudy tap water at certain services in Los Angeles were believed to be caused by galvanic action on the zinc coating of the house service pipe, resulting from a couple formed by the copper pipe on the corporation side of the meter with the galvanized house service. A design for preventing this action has been developed in which the meter is insulated from the galvanized pipe by semi-hard rubber parts placed in the pipe connection.

*Groined Arch Roof for Reservoir at Corpus Christi, Tex.*—Thirty to forty years ago large areas of slow sand filters and reservoirs were covered by plain concrete groined arch roofs arranged in squares or rectangles. Forms were used repeatedly and good economy resulted, but the march of steel in concrete and the general abandonment of slow sand filters caused flat reinforced concrete roofs to be substituted in filter and reservoir work. A circular 10 million gal reservoir has been constructed recently at Corpus Christi in which a groined

arch roof arranged in equilateral triangles has been employed. Very light steel reinforcement was used across groin lines and for temperature.

*Three-Story 10.5 Million Gallon Reinforced Reservoir at Nantes, France.*—Some very fine and durable reinforced concrete tanks and reservoirs exist in France. The largest of these French reservoirs known to your Committee is a 10.5 million gal, three-story, reinforced concrete reservoir recently completed at Nantes. The stories are of different diameter, the lower one being largest and 243 ft in diameter; the second story, 216 ft in diameter, and upper story 179 ft. The side walls of the lower story consists of fifty inclined troughs or inverted segmental arches supported on buttresses. Side walls of the other stories are vertical troughs. Floors are segmental arches radiating from the center. There are many columns carried up in the same lines from story to story. A central standpipe with gates permits operation of each story separately. The entire job appears very complicated to an American, but we may be confident that the pressures afforded by the reservoir are adapted to the service required and that the reservoir will be tight and durable; also that economic considerations were not neglected in selecting this type of construction.

*Elevated Tank Construction.*—Although one of the large tank manufacturers advises that its production of elevated tanks less than 100 000 gal in capacity is not more than 20% of what it was a decade or two ago, there has been a marked increase in the number of elevated tanks of half million to two million gallons. The type of construction of these large tanks has changed almost entirely in the past three or four years from riveted to welded joints. This change in type of construction has also resulted in marked improvement in appearance such as afforded by tubular instead of latticed columns, and rounded roofs largely used for storage instead of simply a cover.

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DISCUSSIONS

# INCREASING THE TRAFFIC CAPACITY AND SAFETY OF THOROUGHFARES

## A SYMPOSIUM

### Discussion

BY MESSRS. LEWIS W. MCINTYRE, AND ARNOLD H. VEY

LEWIS W. MCINTYRE,[35] M. AM. SOC. C. E. (by letter).[35a]—The primary purpose of this paper was to record, for the use of other engineers, certain factual data concerning the actual movement of city traffic. Although these data were obtained possibly as long ago as 1928, for the purpose of establishing whether a signal system could be used in down-town Pittsburgh, they have been found useful in comparing the traffic capacity of various plans for separation of grades, for bridge plazas, traffic by-passes, etc. In a number of cases, the decision as to which of a number of suggested plans should be selected has depended on an analysis of traffic capacity based on these data.

To the best of the writer's knowledge, no similar data have been published. The technique for determining traffic capacity, which has been developed from these data, has been found extremely useful. It is hoped that other engineers may also find the data of service.

ARNOLD H. VEY,[36] ESQ. (by letter).[36a]—In this closing discussion, it is the writer's intention to discuss the comments expressed by Messrs. Lavis, Kelly, and Simpson, all of whom more or less took the writer to task for the optimism he expressed concerning the favorable reduction in accidents which might be expected following major highway improvements.

Mr. Lavis states that the statement to the effect that highway improve-

NOTE.—The Symposium on Increasing the Traffic Capacity and Safety of Thoroughfares was presented at the meeting of the City Planning Division, Pittsburgh, Pa., October 15, 1936, and published in November, 1937, *Proceedings*. Discussion on this Symposium has appeared in *Proceedings*, as follows: February, 1938, by Messrs. John A. Oakey, L. W. Mahone, Fred Lavis, Hugh A. Kelly, and Sidney J. Williams; and in May, 1938, by Messrs. Harold Nevin Carey, W. W. Crosby, Roger L. Morrison, W. R. Flack, Hawley S. Simpson, and Charles M. Noble.

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[35a] Received by the Secretary November 9, 1938.

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[36a] Received by the Secretary January 24, 1939.

ments might do away with 75% of the accidents which now occur "does not seem to be borne out by the facts presented." Mr. Simpson makes the same general statement and uses the words "seems to be unwarranted on the basis of available data." Colonel Kelly is of the opinion that safe road construction is not the solution of the problem and disagrees with the conclusion, contained in the original paper, as to "the impossibility of supervising and governing the actions of all persons at all times." Mr. Lavis considers the real cause of traffic accidents to be the careless and incompetent driver.

The driver, of course, is the cause, and upon him can be placed the largest share of responsibility. After all, of the three elements concerned—the highway, the car, and the highway user—he is in control of the movement of the vehicle; and, too, he shapes the path of the vehicle along the public highways. Therefore, if all highway users were perfect and at all times drove in a manner having regard for the conditions of the vehicle as well as highway conditions, all would agree that there would be no traffic accident problem. However, when indicting the highway user as the element of greatest responsibility, it is necessary to consider all highway users and not any one particular class or group as Mr. Lavis advocates.

It is true that in any one year, a rather small proportion of the licensed drivers (ranging from 10% to 15%) are involved in all the accidents of that year. Unfortunately, however, with few exceptions, the citizens identified in this small group are not responsible for the highway ills year after year. If this were the case, the solution would be rather simple as it would not be difficult to identify these accident breeders quickly and either correct their habits or remove them from the roadway entirely by revoking their driving privileges.

Those involved in motor-vehicle accidents come from all walks of life. For the most part, the accident problem concerns average, law-abiding citizens, many of whom have driven for a number of years and have never before had an accident and, of course, who did not intentionally drive in a manner to cause an accident.

The solution of the motor-vehicle accident problem, despite the complication of causes, can be summed up in two words—"safeguards" and "safe practices"; that is, the application of adequate and appropriate safeguards and the performance of safe practices on the part of highway users.

Safeguards include the construction of highways that are inherently safe—highways that render the failures of human beings of lesser or little importance; and, highways that make it less possible for drivers to perform improper practices, causing accidents. Unfortunately, there are only a few examples of such types of construction in the United States. However, there are many examples of parts of a highway embodying all of the principles and features in one form or another but not confined to, and along, the same section of roadway.

An indication may be had, however, of the probable reduction in accidents through such highway improvements by reviewing accident experience along present-day roadways and determining, from such an analysis, the percentage

of accidents that might be eliminated had the roadway system embodied all of the present-day safeguards known to the highway engineer.

For example, the accident experience along the New Jersey State Highway System over a period of one year was segregated into the various types of accidents and further separated by their place of occurrence—whether at or between intersections—and the following distribution was shown:

| Type of collision | Percentage of accidents at intersections | Percentage of accidents between intersections |
|---|---|---|
| Right angle | 14 | 1 |
| Same direction | 10 | 20 |
| Opposite direction (head-on) | 3 | 18 |
| Fixed object | 2 | 9 |
| Pedestrian | 4 | 6 |
| Opposite direction (left turn) | 5 | 1 |
| Other types | 2 | 5 |

Reviewing the foregoing tabulation, it is apparent that by controlling the right of way (prohibiting acess to abutting property as well as eliminating all cross traffic at grade), mostly all right-angle and opposite-direction (left-turn) types, as well as pedestrian accidents—all of which aggregate 31% of the total—would be eliminated.

The center island, segregating opposing flows of traffic, would prevent nearly all head-on collisions which comprise 21% of the total; fixed-object accidents, equaling 11% of the total, would also be curtailed through the elimination of fixed objects within the travelable part of the highway, together with the elimination of fixed objects adjacent to, and along, the traveled way.

The classification "same direction" concerns three more or less common types of collisions involving vehicles traveling in the same direction—"rear-end," "cutting-in," and "side-swipes." Of these three, the first is by far the most common. Although there has not yet been developed a practical physical means of dealing effectively with accidents of this type, particularly those occurring at between-intersection locations, nevertheless it is the opinion of the writer that the type of highway improvement with which this discussion is concerned would not only correct most of the same-direction accidents that now occur at street intersections but would also affect, favorably, the same-direction accidents that occur between intersections. Furthermore, the application of proper and adequate highway lighting would curtail night-time accidents effectively, particularly those of the same-direction type, many of which occur during hours of darkness, primarily because of insufficient visibility.

It was through a detailed analysis such as this that the writer arrived at the probable 75% reduction in accidents experienced to-day, through proper highway improvements. Neither Mr. Lavis nor Mr. Simpson offers any factual data to support their contentions.

Further indication of the favorable effect of roadway design on present-day accidents may be shown from a comparative study of the accident rate on a million-vehicle-mile basis along a section of an important highway in New Jersey that contains most of the advanced features herein concerned (with

the exception of a center island dividing opposing flows of traffic) with sections of a surface roadway not embodying any of these characteristics.

The accident experience for a period of one year along State Highway Route No. 25 (U. S. Route No. 1, including the 12th Street viaduct, the depressed highway and the Pulaski Skyway" which represents the main approach to the Holland Tunnel, equaled 5.71 accidents per million vehicle miles, whereas for three sections of a surface route—State Highway Route No. 26 (U. S. Route No. 1) in South Brunswick, Plainsboro, and West Windsor Townships—the accident experience during the same year equaled 10.12, 10.88, and 10.34, respectively. A center island along the viaduct, the depressed roadway and the Skyway, would produce further reductions in accident experience.

Another indication of the benefits derived from a controlled highway is the accident experience along the Merritt Parkway in Connecticut, opened to vehicular traffic in June, 1938. Accident experience for the first few months of operation indicates a rate of 0.13 accidents per million vehicle miles of travel as compared with an average rate of 3.1 for the entire highway system of New Jersey.

Both Mr. Lavis and Mr. Simpson are of the opinion that, with all hazards removed from the roadways, motorists will be (as Mr. Simpson states), "lulled into an entirely false sense of security," and that the average speed of the vehicles will increase materially.

Speed surveys made along sections of improved highways, in New Jersey, such as four-lane divided roadways, do not support this contention. It is true that these roadways do not contain all the features of a controlled highway but intersection frequency is such that there is little, if any, border interference. The average speeds of vehicles over these roadways are not substantially different from the speeds along other roadways of the State, other conditions being equal.

In support of his contention, Colonel Kelly cites the experience in the Holland Tunnel as an example of effective enforcement. Since its opening, the Holland Tunnel has had an admirable accident record—a record for which the tunnel authorities can justly be proud. However, in the humble opinion of the writer, this record is due not alone to extraordinary enforcement activities plus the "no passing" regulation but, also, in no small measure, to the elimination of usual conflicts normally experienced or possible along ordinary present-day highways. In other words, the Holland Tunnel is, in a practical sense, a controlled highway—a highway designed for safety.

Colonel Kelly also states that since the passage of an ordinance prohibiting commercial vehicles from using the Pulaski Skyway, and since an increase in enforcement activities along the Pulaski Skyway, fatalities have ceased. The accident records along the Skyway for the five years, 1934 to 1938, inclusive, do not bear out this statement. As a matter of fact, in this 5-yr period, twelve persons were killed as a result of motor-vehicle accidents within the boundary lines of Jersey City and Kearny.

The history of industrial safety is somewhat parallel to the present highway accident situation. When industrial accidents became sufficiently extensive

to warrant attention, industrialists first attempted prevention through a program based entirely on worker-safe practices and completely ignored machine safeguards.

The meager results soon determined that the cart was before the horse, and it was not until proper and adequate safeguards were applied that industrial accidents declined appreciably. In 1938 industrial accidents were less than half of the number twenty-five years ago.

As stated previously, the solution of the motor-vehicle accident problem is dependent upon the application of proper and adequate highway safeguards plus the use of safe practices on the part of highway users. Admittedly, the reconstruction of streets and highways in the manner herein discussed (that is, the rebuilding of highways to eliminate all possible conflicting or interfering flows of traffic by physical separation or segregation) would be economically impossible. However, if the public is to avoid the catastrophes occurring daily along roadways of the United States, it is essential that roadways of the future include design features which automatically lessen the importance of human failures.

AMERICAN SOCIETY OF CIVIL ENGINEERS

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DISCUSSIONS

# GREAT LAKES TRANSPORTATION

## Discussion

By Messrs. Harland C. Woods, W. L. R. Haines, and L. C. Sabin

Harland C. Woods,[7] M. Am. Soc. C. E. (by letter).[7a]—In connection with this paper, it is of interest to note how the developments in bulk freighters have influenced the design of locks at Sault Ste. Marie, Mich. In the early days, freight carriers were general cargo ships suitable for navigating the relatively shallow connecting channels of the central Great Lakes. With the development of the ore, grain, coal, and limestone traffic, the bulk freight carriers were gradually evolved. At first, the dimensions of vessels were gradually increased in conventional ocean-going craft proportions. As the bulk freighter type of vessel emerged, widths were limited by the distance from the face of the dock within which it has been found economically practicable for the unloading machines to operate. This has been found to be less than 70 ft. Structural strength of the vessel limits the depth of hull proportionally to the width. For large vessels the maximum has been found to be about 32 ft. With width and depth defined, the length of vessels has been limited by structural requirements and practical thickness of deck plates to about 600 ft to enable a fully loaded vessel to "ride out" Great Lakes storms safely.

The grain and ore movement involves passage through the St. Marys River and the locks at Sault Ste. Marie. As vessels increased in size, larger locks were built. Fig. 2 indicates, chronologically, the parallel development of vessels and locks. Experience has shown that the lockage of more than one vessel at a time is most expeditious and safe if vessels are tied stem to stem in the lock rather than side by side. Hence, the latest locks built are slightly wider than the maximum width of vessel and long enough to accommodate two of them. Until new developments in trade, vessels, and cargo handling equipment appear, there is little prospect of changes in dimensions of vessels and locks.

Note.—The paper by M. C. Tyler, M. Am. Soc. C. E., was published in November, 1938, *Proceedings*. Discussion on this paper has appeared in *Proceedings*, as follows: January, 1939, by T. Kennard Thomson, M. Am. Soc. C. E.

[7] Senior Engr., U. S. Engr. Office, Buffalo, N. Y.

[7a] Received by the Secretary January 23, 1939.

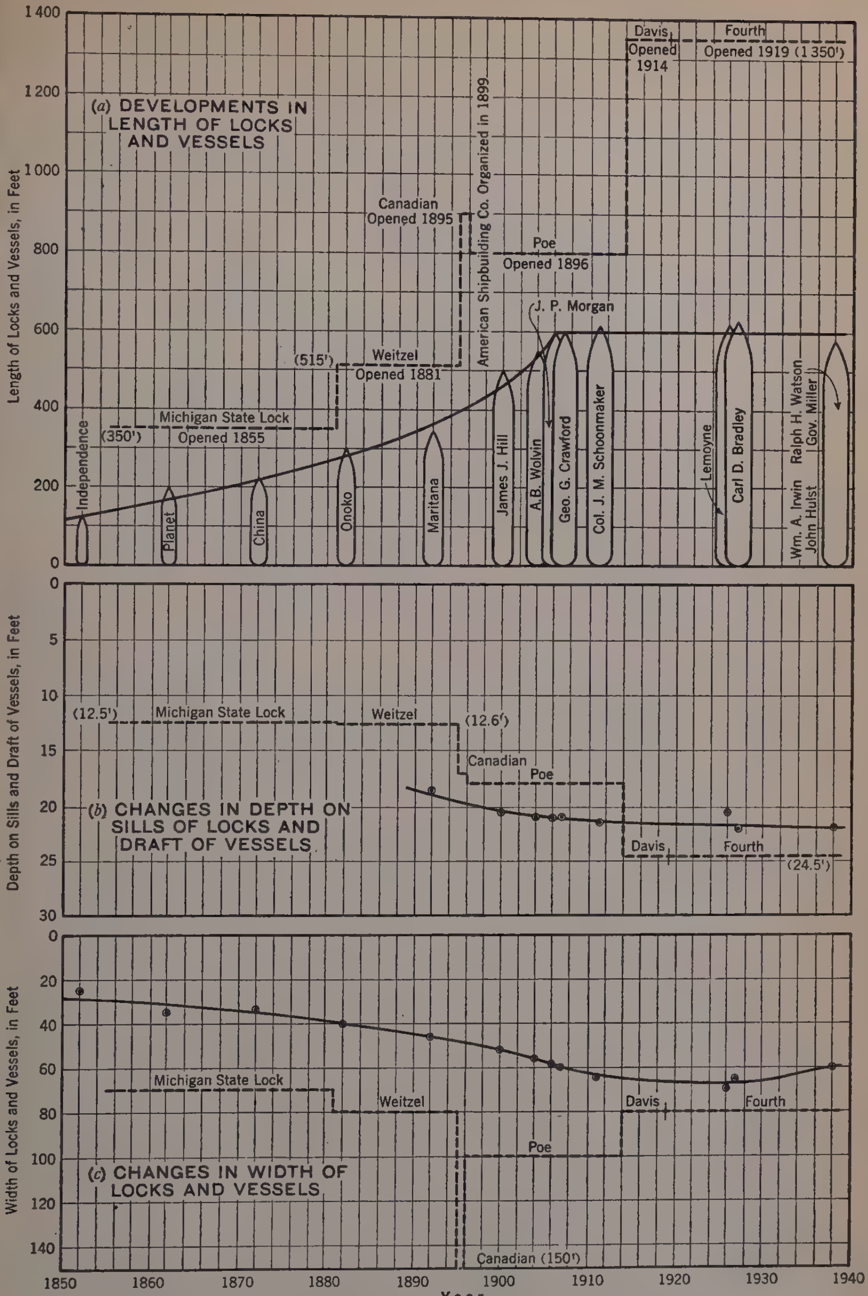


FIG. 2.—CHRONOLOGICAL DEVELOPMENT IN DIMENSIONS OF LAKE BULK FREIGHTERS AND LOCKS AT SAULT STE. MARIE, MICH.

W. L. R. HAINES,[8] M. AM. SOC. C. E. (by letter).[8a]—An able and concise presentation of an historical and statistical review of transportation on the Great Lakes has been presented by General Tyler.

One factor contributing very largely to the efficient and economical development of freight transportation—particularly of ore and coal—on the Lakes is deserving of special emphasis; that is, the great flexibility of the transportation system, combining water haul with rail haul. This is made possible by the development of numerous transhipping docks along the southern shore of Lake Erie, by the railroads. Such facilities offer shippers a wide range of choice as to rail routes between Lake Erie and the consuming or producing districts. They make it possible to divert vessels or cars from one transhipping dock to another in order to avoid delays to vessels which might result from congestion or break-down of facilities at a given dock.

Those interested in the location and ownership of these docks are referred to a report prepared by the Board of Engineers for Rivers and Harbors in 1937.[9] This and an earlier edition of the same report (dated 1930) contain maps devised to illustrate the flow of ore and coal between Lake Erie and the consuming and producing districts.

For many years there has been agitation for the construction, by the Federal Government, of a canal between Lake Erie and the Ohio River, the most favored route being from a point near Ashtabula, Ohio, by way of Youngstown, Ohio, and the Beaver River to a point near Beaver, Pa. The cost of constructing this canal has been estimated by the Government engineers at about 203 million dollars. This sum, however, does not include facilities and equipment for operating and maintaining traffic on the canal; nor does it include the entire cost of rearranging railroad bridges and tracks made necessary by its construction. Experienced engineers have estimated that the total cost would be about 281 million dollars.

The principal purpose of the canal would be the transportation, in barges, of ore from Lake Erie and of coal from the Ohio River. This tonnage is now carried by the railroads and would be diverted from them. One effect that would result would be an impairment of the flexibility of operations on the Great Lakes, and of the service of the railroads.

It is inconceivable that the railroads could afford to maintain all of the transhipping docks now in existence. Those at Ashtabula Harbor and Fairport Harbor, Ohio, would almost certainly be abandoned, because of the diversion of tonnage to the canal. In the opinion of experienced railroad men in positions to judge accurately, 17 ore unloaders, 3 ore docks with supporting storage, 4 coal unloaders, and possibly 1 grain elevator and 1 warehouse at Lake Erie ports would be abandoned for the same reason. With the abandonment of these facilities, the opportunity for diversion of vessels from one dock to another would be destroyed to a great extent.

Another important factor aiding in the expeditious movement of ore and

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[8] Special Engr., Office of Vice-Pres., P. R. R., Pittsburgh, Pa.

[8a] Received by the Secretary January 13, 1939.

[9] "Transportation on the Great Lakes," prepared by the Board of Engineers for Rivers and Harbors, *Transportation Series No. 1* (revised 1937); published by the U. S. Government Printing Office, Washington, D. C.

coal in Lake transport is the "Ore and Coal Exchange." Mr. E. A. Burslem, Examiner, Interstate Commerce Commission, states[10] that this is:

> "* * * a central agency established and maintained by the railroads with all their individual rights as to operation, dealing with shippers, and customary practices of the individual railroads * * *. The expenses of the Exchange are borne by the railroads * * *." The Exchange is for the purpose "of providing the best possible services to lake shippers; to furnish information as to coal at the ports, in transit, and the operation of coal machines at the docks; to supervise the supply, movement, routing, and delivery of ore to the furnaces; to establish contacts between railroads, coal shippers and lake vessel owners; and to furnish such other services as would facilitate the movement of coal and ore, thereby reducing the detention of cars, increasing the car supply, and adding to the facilities of the lake traffic."

Mr. Burslem cites the following as some of the more important functions of the Exchange:

(1) To issue permits regulating the movement of the lake coal to the ports; (2) to receive reports from all docks west of Buffalo of coal at the ports and the number of cars of coal in transit and due in the ports in the following 12-hr, 24-hr, 36-hr, and 48-hr periods for each transhipper or consignee, with the number of cars in each consignment; (3) to issue daily reports showing the quantity of coal at each dock, the quantity in transit, previous day's dumping and estimated dumping for the current day; (4) to keep account of the various vessels unloading at the Lake Erie ports which are named for coal loading, and to ascertain the time the boats will be unloaded and ready to sail, so that the dock at which the vessel will receive its lake-coal cargo will be advised of the time of the vessel's probable arrival in order to get coal to the docks and not delay the loading of the vessels; and (5) to receive orders from transhippers for loading coal into vessels named by the transhippers, and to transmit such information to the docks. The Exchange keeps all interested lake-coal shippers, transhippers or consignees, and carriers informed as to the situation, daily, so that they may co-ordinate their efforts and thus effectively permit the unloading of cars and the dispatching of vessels.

Many of the vessels transporting coal from the lower Lake Erie docks come to those docks loaded with ore. Thus the speed of "turn-around" is affected by both the unloading and loading time. The provision, by the railroads, of ground storage for enormous quantities of ore makes it possible to discharge the ore cargoes even if there is not immediate demand for shipment to the furnaces. Furthermore, this ground-storage makes possible the shipment of ore to the furnaces as it is needed after the close of the Lake season.

The reduction of the average stay of iron-ore carriers at Lower Lake ports from 36 hr (in 1906) to less than 11 hr (in 1937), mentioned by the author, is largely due to the capacity and efficiency of the docks owned by the railroads, the efficient service of the railroads themselves, and the service of the Ore and Coal Exchange, maintained by and at the expense of the railroads.

The writer objects to the implication which might arise from the comparison of Lake rates and rail rates on ore, given by the author. The statement is

[10] Proposed report in Interstate Commerce Commission Docket No. 27266.

made that the "water haul" is $0.98 per gross ton from port to port as against $1.96 for the two rail hauls (that is, from the mines to the Lake and from the Lake to destination) for ore delivered at Pittsburgh and $1.63 for ore delivered at Youngstown (see heading, "Freight and Passenger Traffic: Ore"). The inference might be drawn that rail rates were excessive. Such a comparison is scarcely fair.

The author states (see "Cost and Savings") that the total expenditures for new work by the War Department on the channels, canals, and harbors of the Great Lakes to June 30, 1937, were $193 573 686.83. In addition to this, the report of the Chief of Engineers, U. S. Army, for 1937, lists an expenditure of $66 940 948 for maintenance and of $174 061 "not distributed"—a total of $67 115 009 for the five Lake engineer districts (that is, Chicago, Milwaukee, Duluth, Detroit, and Buffalo) to the same date. Thus the total expenditure amounts to $260 688 696, making an annual interest charge, at 5%, of $13 034 435. During the decade 1926 to 1935 the average annual commerce of the United States Great Lake ports is stated by the author to have been 114 837 300 short tons (see "Freight and Passenger Traffic"). The resulting cost amounts to $0.114 per short ton.

Reducing the charges per gross ton given by the author to terms of short tons, and adding to the water haul the cost (borne by the taxpayers) per short ton resulting from Government expenditures, gives the following:

| | | |
|---|---|---|
| Water haul charges | $0.875 | |
| Interest on Government expenditures | 0.114 | |
| Total | | $0.989 |
| Rail Haul Charges: | | |
| Mines to Lake ports | | 0.741 |
| Lake ports to Pittsburgh | | 1.027 |
| Lake ports to Youngstown | | 0.732 |

It may be assumed that the sums reported as spent for new work and for maintenance are actual costs, without any addition for interest. Had interest been added to actual expenditures from year to year and the resulting totals carried forward to show the total cost to any given date, the charges would be vastly increased.

There should also be added the costs to the Government incurred by the U. S. Lake Survey, the U. S. Bureau of Lighthouses, the U. S. Coast Guard, and by the U. S. Weather Bureau in its operation of storm-warning stations and forecasts for the Lakes region, especially applicable to navigation. These services are all mentioned by the author, but without including any costs.

It is the opinion of the writer that it is scarcely proper to combine the two rail hauls; that is, from the mines to the Lakes and from the Lakes to destination in the manner done by the author. These two steps are performed under very dissimilar conditions.

The Board of Engineers, cited previously,[8] called particular attention to the contribution of railroads in the rapid development of the various ore ranges (or mines). A special type of carrier was developed in order to handle

ore from mine to vessel economically and with the greatest dispatch. The fact that there is a wide divergence in the chemical constituents of ores makes sampling and mixing necessary in order to create a certain grade. Furthermore, the ore vessels must be loaded promptly with a fixed tonnage, the chemical content to be always within a small fraction of 1% of the analysis on which it was sold. All of these factors, according to the Board, "do not allow much flexibility in the rail transportation of ore."

The special equipment provided, and the complexity of movement from the mines to vessel, mean additional expense, which must be taken into consideration in any comparison of rail rates. The movement of ore from the lower Lake Erie ports to the mills is less complicated. Special equipment is not required, but the movement is through a more congested district, and requires close supervision by the railroads to insure prompt delivery to the mills.

The operations of blast furnaces require specific mixes of ore which must be met by shipment of the grades of ore as needed. Many of the mills have little or no storage space and must depend upon incoming shipments to meet day-to-day requirements. Frequently it happens that emergency movements of ore must be made by the railroads to meet the demands of the mills.

The maintenance and operation of the facilities required for the movement of a cargo from Lake Erie to its destination are all items of expense in addition to the "line haul cost" itself, but they are included in the rate charged. On the other hand, the vessel movement from port to port is a comparatively simple one. Once the cargo is loaded, the vessel makes a continuous voyage to its destination, carrying, in some instances, nearly 15 000 gross tons of ore. For the years 1928 to 1936, inclusive, the average cargo was 9 331 tons, and the average maximum cargo 13 489 tons.

Data are not available to the writer to show the total expenditures by the railroads in providing transhipping facilities. However, the report of Mr. Burslem[9] shows an Interstate Commerce Commission valuation of the coal-loading facilities alone, as of December 31, 1935, with depreciation allowed, amounting to more than $13 000 000. To this sum should be added the value of the ore-unloading facilities, of the ore-storage areas, and of the supporting yards for both ore and coal.

The writer does not take any issue with the "Conclusion" stated by the author. He merely wishes to emphasize the part played by the railroads in the development of Lake transportation, to indicate certain "hidden costs" and to anticipate any erroneous conclusion that might be drawn as to relative costs to the public of Lake or rail transportation.

L. C. Sabin,[11] M. Am. Soc. C. E. (by letter).[11a]—The co-ordination of effort by the United States, the shipping interests, and the railroads to build up a most efficient system is emphasized in this paper. Not since the Annual Presidential Address[12] of the late Alfred Noble, Past-President, Am. Soc. C. E., in 1903 has the subject been covered with such thoroughness and understanding.

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[11] Vice-Pres., Lake Carriers' Assoc., Cleveland, Ohio.

[11a] Received by the Secretary January 27, 1939.

[12] *Transactions*, Am. Soc. C. E., Vol. L, June (1903), p. 327.

Little comment remains to be made, and that only to emphasize some details regarding the extent and value of the system.

The author defines the value of water transportation in times of emergency. Even in the French and Indian Wars and in the struggle of 1812, before any improvements had been made, the waterway served a military purpose. In the Civil War the supplies for armament were made available by water service, whereas in the World War a stoppage of the flow of commerce on the Great Lakes, involving food as well as raw materials for munitions, would have been most serious. The comparative isolation now enjoyed by the system is not without its value as a feature of defense in future possible emergency.

Bearing in mind the contribution of this system of transport, attention may be called to the fact that the expenditure of the United States in improving channels and harbors—less than 200 million dollars—is equal to the cost of only three battleships; and, while it is held in readiness during peace time, for service in emergency, the system is returning annually in savings the full cost of the development.

The author mentions the practice, followed in channel improvement, of keeping the up-bound and down-bound channels separate wherever possible. A similar system is in effect in the open lakes where on Lakes Superior, Michigan, and Huron the Lake Carriers' Association has designated separate lanes for up-bound and down-bound traffic. Adherence to these lanes is general on the part of Lake Carriers' ships, and many others follow them habitually. Their use has resulted in greatly reducing the chances of collision.

The lengths of the sailing courses from Duluth and Chicago to Buffalo are 986 miles and 893 miles, respectively. Deep-water navigation extends to Ogdensburg, N. Y., and Prescott, Ont., Canada, on the St. Lawrence River, adding 230 miles to these distances. The improved reaches in the connecting channels total about 139 miles in length.

Readiness of the ship operators to co-operate with the United States is shown by the fact that, as the improvements progressed, vessels were built to take full advantage of the better facilities. Prior to the opening of the Poe Lock in 1896 there were no cargoes exceeding 5 000 tons. Nine years later there were 191 vessels that carried more than 5 000 tons, and cargoes of 12 000 tons were being carried. Prior to the opening of the Davis Lock about 14 000 tons was the maximum cargo, whereas subsequently, cargoes of 17 000 tons were carried.

The U. S. Lighthouse Service has contributed generously to the safety of navigation. More than 900 lighted aids and more than 200 fog signals are maintained. There are also in operation 41 radio-beacon stations at important points, and all vessels enrolled in the Lake Carriers' Association are equipped with radio direction finders, the most important recent contribution to safe navigation.

Radio telephones are in use on 80 United States vessels and 50 of Canadian register. These have proved so satisfactory as a means of communication from ship to shore and ship to ship that their general use seems only a matter of time. The U. S. Lighthouse Service in co-operation with the U. S. Weather Bureau, U. S. Coast Guard, and U. S. Hydrographic Office broadcasts information regarding weather and other matters affecting navigation; and the U. S. Coast

Guard Service contemplates full co-operation through supplying radio equipment to a large number of life-saving stations on shore.

In the further interest of safe navigation load lines are marked on all ships limiting the depth to which they may be submerged. In channels of limited depth current drafts are recommended by the Lake Carriers' Association according to the progress of dredging and the prevailing water level.

Among the ports on the Great Lakes there are 18 having an annual traffic in excess of 3 million tons. On the Atlantic Coast there are 13 such harbors, and 7 on the Pacific. The net total United States Commerce of the Great Lakes, excluding that pertaining only to Canada, has averaged 117 million tons annually for the ten years 1928–1937. The foreign commerce of all ocean and gulf ports for the same period averaged 81 million tons. When computed in ton-miles of movement the traffic is still more impressive. In 1936 the ton-mileage on the Lakes was 77 billion, whereas that of all other inland waterways was 15 billion. Another view of the volume of traffic may be gained by the statement that during the seasons of navigation covered by the six years from 1924 to 1929 ships passed Detroit on the average of once every 12 min with an average cargo of 3 300 tons.

Freight rates are given for ore and coal. Reduced to net tons the cost of ore transport, exclusive of terminal charges, is 71.4 cents per ton. The length of haul varies from 280 miles (a small tonnage from Escanaba to the south end of Lake Michigan) to 986 miles (Duluth to Buffalo) with an average of about 800 miles. The ton-mile rate is thus about 0.9 mill.

On coal from Lake Erie to Duluth the rate is 45 cents per net ton for 800 miles, which gives a ton-mile rate of 0.56 mill. The rate to Milwaukee on Lake Michigan is 55 cents (a trifle higher than Lake Erie to Duluth because of lack of return loads), giving a ton-mile rate of about 0.65 mill. Shorter hauls from Lake Erie to near-by ports carry a higher ton-mile rate. The combined rates on 87 million tons passing St. Marys Falls Canal in 1937, including package freight, is given as 1.07 mills per ton-mile. This low cost of transportation is not found in any other large transportation system.

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DISCUSSIONS

# TRANSPORTATION OF SAND AND GRAVEL IN A FOUR-INCH PIPE

## Discussion

BY H. S. GLADFELTER, M. AM. SOC. C. E.

H. S. GLADFELTER,[36] M. AM. SOC. C. E. (by letter).[36a]—Dredge engineers and others interested in the hydraulic transportation of solids will find this paper valuable because it furnishes information on a subject about which there is very little published.

The data and conclusions arrived at by the author have been obtained from laboratory tests under conditions that were controlled during the entire test period. The pipe being tested was straight, of uniform cross-section, smooth on the inside; and it was possible to control the velocity, grain size, and percentage of solids. Compare these almost ideal test conditions with those encountered in testing large dredges engaged in actual dredging operations. In the latter case, long pipe lines, seldom straight, are made up in sections 50 ft to 60 ft long, connected by rubber sleeves, ball joints or tapered joints, which introduce either contracted or expanded sections in the pipe lines. Furthermore, the percentage of solids, pipe-line velocity, and character of the material may vary greatly within short periods. The presence of trash in the system intake also may cause test periods to be interrupted. These conditions render any test to determine pipe-line characteristics very difficult and may account for the meagerness of published information on the subject.

During the three years, 1936, 1937, and 1938, the writer made a number of tests on 32-in. pipe-line dredges of the dust-pan type operated by the Memphis Engineer District on the Mississippi River, in the maintenance of its navigable channel. They included the determination of the carrying capacity of several types of pipe lines. The observations made during these tests were suggested by the author, and were generally similar to observations made on the 4-in.

NOTE.—The paper by George W. Howard, Jun. Am. Soc. C. E., was published in September, 1938, *Proceedings*. Discussion on this paper has appeared in *Proceedings*, as follows: December, 1938, by Messrs. Fred R. Brown, Joseph E. Montgomery, Elliott J. Dent, and David L. Neuman; January, 1939, by Morrough P. O'Brien, M. Am. Soc. C. E., and R. G. Folsom, Esq.; and February, 1939, by Messrs. R. L. Vaughn, M. P. Durepaire, and Pierre F. Danel.

[36] Senior Engr., U. S. Engr. Office, Memphis Dist., Memphis, Tenn.

[36a] Received by the Secretary February 1, 1939.

test pipe at the U. S. Waterways Experiment Station. A comparison by the author of the test results obtained in the laboratory with those obtained in the field on the 32-in. dredge should furnish valuable information.

In Fig. 4 the author shows the distribution of sand in the 4-in. test pipe. The distribution of sand in the 32-in. pipe tested is shown in Figs. 16(*a*) and 16(*b*). This condition was found to exist with pipe-line velocities of 14.78 and

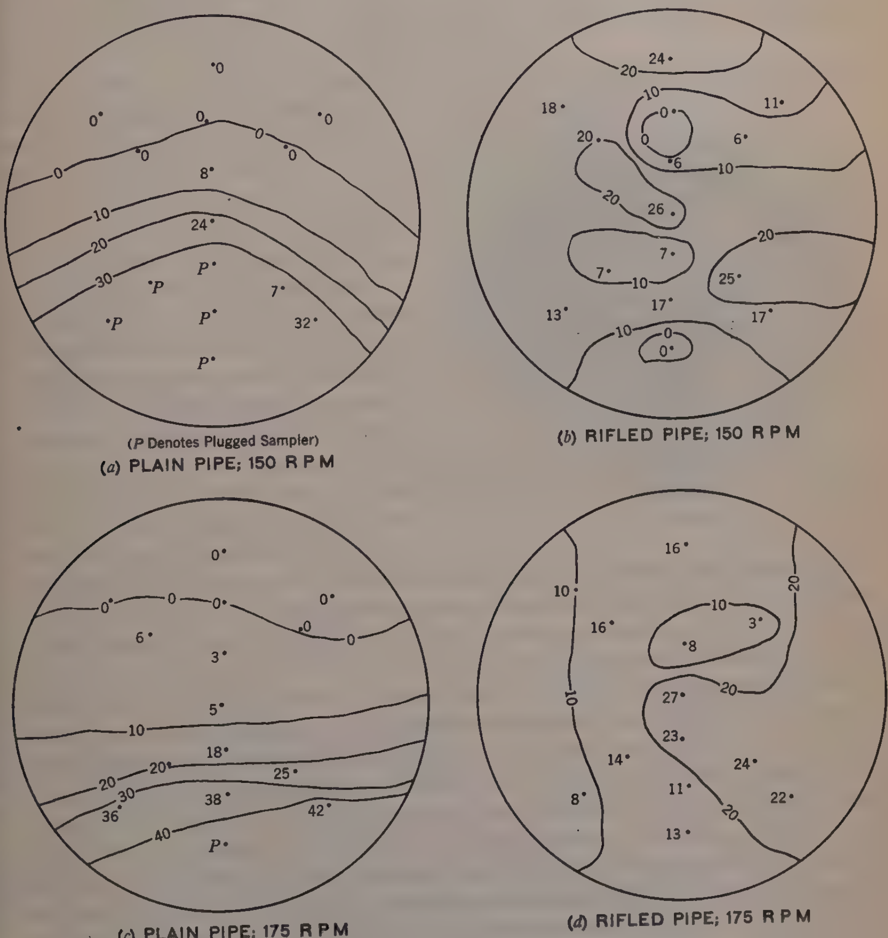


FIG. 16.—DISTRIBUTION OF SOLIDS IN 32-INCH PIPE

21.85 ft per sec, respectively, and the observed percentage of solids under these conditions was 13.3 and 18.2, respectively. Examination of the distribution of sand in the two sections of pipe shows that the sand is transported in the manner described by the author as the first method. The writer has found that low pipe-line velocities will result in plugged pipe lines. The earlier dredges built by the Mississippi River Commission were operated at pipe-line

velocities ranging from 13 to 18 ft per sec with resulting percentages of solids ranging from 10 to 15. When these dredges were operated at a high rate of advance, with increased percentage of solids pumped, frequent plugging of the pipe line resulted. In dredges now (1939) operated by the Memphis District, velocities of 25 to 26 ft per sec pumping water, and 23 to 25 ft per sec pumping solids, are obtained.

To improve the carrying capacity of pipe lines used for transporting heavy material the Memphis District installed rifles or vanes on the inside of the pipe line of one of the dredges of the district, and included tests of this type of pipe with tests of plain pipe. The effect of the rifles was to reduce the concentration of solids in the bottom of the pipe and prevent plugging that may occur at low pipe-line velocities. These rifles are shown in Fig. 17. The test of this type of pipe showed concentration of solids as in Figs. 16(*c*) and 16(*d*). These tests were made at the same pumping speeds obtained in tests that produced the concentrations of solids shown in Figs. 16(*a*) and 16(*b*). In this test the pipe-line velocities were 15.79 and 20.30 ft per sec, respectively, and the average percentage of solids was 18.1 and 24.7 respectively. This distribution of solids in the rifle-fitted pipe is more uniform than in the plain pipe, and is a result that may be expected in the third type of transportation of sand referred to by the author.

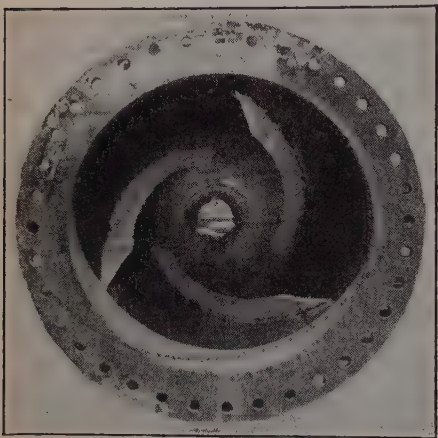

FIG. 17.—TYPICAL RIFLED PIPE

Mr. Howard has referred to the "economical velocity" for sand transportation, and gives four factors that must be balanced carefully for its determination. This procedure may be permissible when a new dredge is being designed. After its construction, however, the pipe-line velocity can be changed within limits by changing the speed of the prime mover or by changing the pump impeller. One condition which is not referred to by the author in determining the economical velocity and which would appeal to dredge operators is the cost of pipe-line and pumping equipment maintenance. When pumping a high percentage of solids containing abrasive material, the wear on the pipe line, pumps, etc., may be excessive at high velocities and it may be found more economical to operate at reduced velocities.

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DISCUSSIONS

# A THEORY OF SILT TRANSPORTATION

**Discussion**

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BY S. G. BAUER, ESQ.

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S. G. BAUER,[60] ESQ. (by letter).[60a]—As an attempt to discover the influences determining the shape of a stable channel by studying equilibrium considerations, this paper is exceedingly interesting. No doubt it would be quite impossible to take into account all the factors that might influence the cross-section of a channel so that simplifying assumptions must be made. The author's deductions from his assumptions are certainly most interesting and show that the principle is a step in the right direction. His assumptions, however, cannot be viewed with quite the same confidence.

In a channel of infinite width and constant depth it would be quite justifiable to consider the vertical components of the turbulence eddies only for the upward movement of the silt; but when considering the shape of a finite channel, with sloping walls, the horizontal components of the eddies should also be taken into account, responsible as they are for the horizontal velocity distribution across the channel. This raises another point. If the mean velocity in any particular vertical plane were proportional to the square root of the depth in that plane the horizontal velocity distribution would be determined by the bed contour only. As the horizontal components of the eddies provide a constant interchange of energy from the faster to the slower flowing water, this appears a very unlikely assumption.

Another point is that the mean velocity of the silt charge of a channel is quite different from the mean velocity of the water, because most of the silt is carried near the bottom of the channel where the average forward velocity of the water is only a fraction of its all-round mean velocity. Moreover, the finer silt moves much faster than the coarser silt in the same channel. In consequence, the distribution of the silt charge along a vertical section has a profound influence on the silt-carrying capacity of a channel. The author

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NOTE.—The paper by W. M. Griffith, Esq., was published in May, 1938, *Proceedings*. Discussion on this paper has appeared in *Proceedings*, as follows: September, 1938, by Joe W. Johnson, Jun. Am. Soc. C. E.; October, 1938, by Messrs. George W. Howard, Harry F. Blaney, and E. W. Lane; December, 1938, by Messrs. O. A. Faris, J. E. Christiansen, Samuel Shulits, and Gerald Lacey; February, 1939, by Messrs. Glenn W. Holmes, and Hunter Rouse.

[60] Eng. Laboratory, Univ. of Cambridge, Cambridge, England.

[60a] Received by the Secretary December 13, 1938.

assumes a silt distribution varying with the square of the depth, irrespective of the nature of the silt charge. This may be the case, approximately, with some types of fine sand on a very smooth bed. The coarser grade of silt is lifted from the ground by a large velocity gradient which results in a higher velocity of flow above a particle than below it, thereby imparting to it a "lift" similar to that of an airplane wing in flight. The type of silt carried must therefore enter into the law of its distribution. The writer's experience with fine sand in a tidal river in North Wales, Great Britain, flowing in its own silt bed, indicates that even with fine sand the silt charge is much heavier near the bottom of the channel than would correspond to the square law. That gravel and boulders are carried along the bed of a river or channel only is common experience.

Apart from that, the silt distribution is strongly influenced by the presence, or otherwise, of large eddies of the type which produce an unevenness of the water surface. There seems to be some critical rate of flow above which such large-scale eddies appear, sometimes covering the entire depth of the channel, and thus changing completely the entire velocity and silt distribution. At a rate of flow at which the water still appears quite smooth, but which is near that critical rate of flow, this large-scale turbulence can often be released by a small disturbance, such as can be made by throwing a stone into the water or disturbing the evenness of the bed with a stick.

This critical condition has been observed by many, no doubt, and should be taken into account in a manner similar to the critical Reynolds number for the flow in pipes. Dimensional analysis may ultimately lead to a solution. Where these large eddies appear, which are of the same order of magnitude as the width of the channel, it is obviously difficult to derive any consistent laws, but it may well be that this condition is actually a limiting one for the stability of the channel itself.

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DISCUSSIONS

# LATERAL EARTH AND CONCRETE PRESSURES

## Discussion

By David A. Molitor, M. Am. Soc. C. E.

David A. Molitor,[42] M. Am. Soc. C. E. (by letter).[42a]—After reviewing this paper, wherein it is recommended that the classic theories of earth pressure be discarded without supplying any adequate substitute, one wonders what method should be used in future. Actually, only the special case of lateral pressures due to single or strip-surface loads is treated. The title, therefore, is too inclusive for the subject matter presented.

The authors mention three phases of retaining wall problems, as follows:

(1) Pressures distributed along a wall sufficiently rigid or unyielding so that no fracture surfaces appear in the ground or bank;

(2) Pressures developed as a result of a fracture surface or surface of rupture caused by a slight movement of the wall; and,

(3) The distribution of pressures due to a surface loading. Phase (1) is not treated in detail; Phase (2) is the basis for Coulomb's theory, which the authors recommend be discarded; and Phase (3) is the case analyzed by an application of the Boussinesq-Hertz formulas and comprises most of the paper. The writer has never seen a case that could properly be classed as belonging under Phase (1). Even if the wall structure is perfectly rigid (which condition was more likely to exist in the days when only gravity type walls were in use) the elasticity of the foundation is always sufficient to cause a decided movement in the back-fill, not to mention the ever-occurring settlement.

No wall can be regarded as safe and enduring unless it is so designed that it will stand at least 1 yr without undergoing any serious displacement, either of overturning or sliding on the base. Under such circumstances would any designer be warranted in pursuing any other than a safe course by providing adequately for the possible maximum earth pressure? Hence, whether he

Note.—The paper by Lazarus White and George Paaswell, Members, Am. Soc. C. E., was published in September, 1938, *Proceedings*. Discussion on this paper has appeared in *Proceedings*, as follows: November, 1938, by A. E. Cummings, M. Am. Soc. C. E.; December, 1938, by Messrs. Robert G. Hennes, Robert F. Legget, and Charles Terzaghi; January, 1939, by Messrs. Jacob Feld, M. G. Spangler, and Raymond D. Mindlin; and February, 1939, by D. P. Krynine, M. Am. Soc. C. E.

[42] Cons. Engr., Harlingen, Tex.

[42a] Received by the Secretary December 27, 1938.

likes it or not, the engineer's design problems will continue to be solved according to Coulomb, Poncelet, Scheffler, Rankine, Winkler, etc., on the basis of the prism of maximum pressure, Phase (2), and nothing better has been proposed as yet.

As for Phase (3) (which is merely a special case of surcharge superimposed on Phase (2)) the lengthy mathematical treatment in the paper is interesting but of comparatively little value until an entirely new set of empiric coefficients is determined. This is equivalent to reminding the designer that he must first conduct some elaborate test experiments prior to designing a wall.

The authors state that they hope to present the development of the pressures due to shear failure on the assumption that the failure surface is a cosine curve. Furthermore, a typical section of fracture surface is shown in Fig. 1, and it is stated that this surface is independent of the type of ground. Others have assumed the fracture surface to be of circular cross-section.

The writer contends that the surface of rupture is of hyperbolic section, and that its particular shape and location is purely a characteristic of the type of ground and its moisture content, as described in a paper[43] giving a full account of actual measurements of such failure surfaces for clay embankments as high as 65 ft, with a possible application to the earth-pressure problem.

In 1937 the writer presented in brief form[44] the most usable features of the many theories evolved in the past, and explained how these theories may be applied in a practical manner to retaining-wall and sheet-piling problems. To those designers who still feel that the classic earth-pressure theories are not yet doomed to oblivion, this information should prove valuable.

It is regrettable that the term "equivalent fluid pressure" has crept into modern textbooks. The term could apply only to the case of level-back fill. As most wall problems involve surcharge in some form, the designation is not fitting.

In conclusion it should be emphasized that the design of a large, reinforced retaining wall, with or without surcharged back-fill, requires more than mere textbook knowledge and should be undertaken only after considerable experience and mature judgment are acquired.

Predicated on these qualifications, the classic theories will no doubt endure for quite some time to come, or at least until something much better can be evolved. When one considers the immense number of walls and structures, designed and built during the past, with a relatively small percentage of failures traceable to poor judgment, it must be acknowledged that the classic theories have served well in practice.

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[43] "The Present Status of Engineering Knowledge Respecting Masonry Construction," by David Molitor, *Journal*, Association of Eng. Societies, 1900.

[44] "Structural Engineering Problems," by David A. Molitor, The Peters Co., Detroit, Mich., 1937, Chapters 9 and 10.

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DISCUSSIONS

# WIND FORCES ON A TALL BUILDING

## Discussion

BY MESSRS. ALBERT SMITH, AND VICTOR R. BERGMAN

ALBERT SMITH,[45] M. AM. SOC. C. E. (by letter).[45a]—A large mass of original data is presented in this paper, which should be analyzed carefully to reveal all the information it will yield. From Table 1, the writer took all the items in which the velocity was as great as 30 miles per hr at the Empire State Building, and separated them into 16 compass directions. For each direction the average of the velocities was computed for the Empire State, the Daily News, and the Whitehall Buildings. (It is assumed that all readings have been corrected, and can be compared.) The numbers of readings are plotted in Fig. 24. The approximate effective depths of wind current are as follows:

| Buildings | Depth, in feet |
|---|---|
| Empire State (1 263 − 100) | 1 163, or less |
| Daily News (504 − 100) | 404, or less |
| Whitehall (within a horizontal arc of 247°) | 454 |

The dotted line for the Daily News Building and the full line for the Whitehall Building show ratios of the average velocity in each direction, to the average velocity at the Empire State Building in the same direction. Within an arc of 112° the high buildings near the Whitehall Building evidently reduce the velocity; and at the Daily News Building the influence of the Chrysler Building on the velocity of west northwest winds is clearly visible. Over an arc of 247° the Whitehall Building has a larger wind current depth than the News Building, and averages 0.617% as compared with 0.51% for the News Building. One would not expect so great an increase in velocity between 500 ft and 1 200 ft of height. These data seem to indicate that the idea of using a uniformly increasing pressure, advanced by H. V. Spurr, M. Am. Soc. C. E.,[10]

NOTE.—The paper by J. Charles Rathbun, M. Am. Soc. C. E., was published in September, 1938, *Proceedings*. Discussion on this paper has appeared in *Proceedings*, as follows: November, 1938, by Messrs. David C. Coyle, and Clyde T. Morris; and January, 1939, by Messrs. Robins Fleming, F. P. Shearwood, Lydik S. Jacobsen, Francis L. Castleman, Jr., J. B. Wilbur, R. D. Spellman, David A. Molitor, Walter J. Gray, and K. L. DeBlois.

[45] Pres., Smith & Brown, Engrs., Inc., Chicago, Ill.

[45a] Received by the Secretary December 29, 1938.

[10] "Wind Bracing," by H. V. Spurr, McGraw-Hill Book Co., Inc., 1930.

is better than making no increase above 500 ft. Actually the readings indicate about four times as much pressure at the top of the Empire State Building as at the top of the News Building. It is likely, however, that the "20-yr wind," of much greater velocity than any measured in these tests, will have a

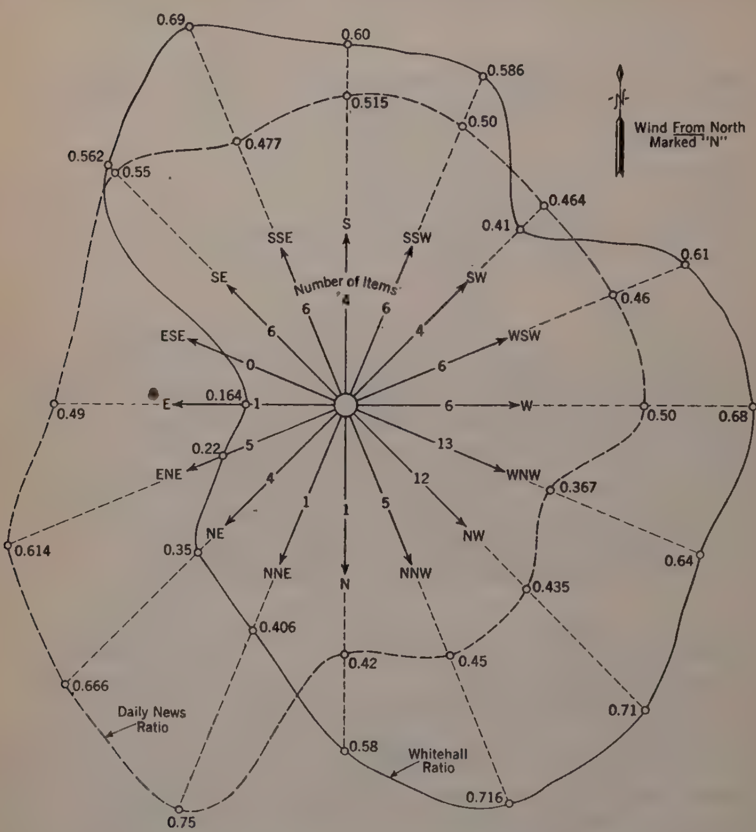


FIG. 24.—WIND VELOCITIES OF 30 MILES PER HOUR, OR MORE, AT EMPIRE STATE BUILDING

velocity more nearly uniform above the 500-ft level. Returning to the consideration of shelter, winds from the south southwest appear to have a clear sweep for a long distance before reaching either the News or the Empire State Buildings, and one might expect the News velocity ratio be be high for this direction, instead of which it is about average. On the whole, there seems to be no evidence that anything except the very high Chrysler Building affects velocities at the News Building, and that therefore the velocities at the Empire State Building have not been reduced by the influence of other buildings.

*Manometer Readings.*—The writer can see no possibility of error in the preparations for these tests. In a multi-story building there may be a difference in the interior air pressure at different stories; but the difference is small,

and would not affect the total force. Since the pipes leading from the wall openings were very different in length, Tubes No. 3 and No. 8 (Fig. 4) would be much more responsive to gusts than the others; but this should not cause large errors. There are a large number of cases in which the approximately simultaneous readings at the stations on the same floor are of magnitude and sign that contradict all previous data from manometer readings. It may be that the 90-mile wind illustrated in Fig. 10 changed 90° between the readings of the two sets of five tubes each, and that this accounts for positive readings at both Tubes No. 7 and No. 4, but the writer cannot account, in Fig. 10(*c*) for readings in a 90-mile wind of simultaneous pressures at Tubes No. 6 and No. 10. He suggests that the sign for Tube No. 10 has been recorded in error. The same comment applies to the reading on Tube No. 10 at 50 to 60 miles per hr (Fig. 10(*c*)).

Of thirty-seven complete sets of observations for the seventy-fifth floor, eleven contain readings which contradict each other, and all sets exhibit slopes in the pressure diagram which differ very greatly from all those previously recorded. The air tunnel results of Messrs. Dryden and Hill[3] are consistent and explainable. Although the structure of the air that blew against the seventy-fifth floor was more variable than that in the wind tunnel at the National Bureau of Standards, it was probably less complicated than that in which the writer's observations were taken on the campus of Purdue University, Lafayette, Ind.[46] When plotted, his results were consistent internally, except for such variations as should be expected in light winds and near the ground, and agreed very well with wind-tunnel tests. For this reason he would like to know much more about the technique of the author's readings before he revises his ideas of how the wind forces are distributed.

*Extensometer Readings.*—Some of these readings correspond to a stress computed for the observed velocity, but many do not. Observations No. 9 and No. 10, Table 6, with the same velocity and direction of wind, show stresses of +223 and −343, respectively. Observation No. 26, during a south southwest wind of 30 miles per hr, showed a column stress of −488 lb per sq in., which would check quite well if it had been +488. Observation No. 28, during a south southwest wind of 18 miles per hr, showed a column stress of +59 lb per sq in., which, although correct in sign, is low in amount.

There is no mention in the description of the insulating precautions. Undoubtedly, some precautions were taken, but it is possible that they were not adequate. The largest fiber stress derived corresponds to about 4° differential in temperature between the rod and the column, and when the effect of a cold wind on this column above the twenty-fifth floor is considered, and the rapidity with which the twenty-fourth-story column would lose heat, it seems clear that the measuring rods should be insulated very carefully, or that exact temperature corrections should be made.

*Plumb-Bob Readings.*—The readings in Table 3 show something that one knows must occur, but which heretofore lacked proof. Fig. 12 indicates a

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3 "Wind Pressure on a Model of the Empire State Building," by Hugh L. Dryden and G. H. Hill, *Research Paper No. 525*, National Bureau of Standards.

46 "Wind Pressure on Buildings," *Journal*, Western Society of Engineers, December, 1912; also, "Wind Loads on Buildings," *loc. cit.*, April, 1914.

"no man's land" under the bob about 1 in. in diameter. The tests prove that the eighty-sixth floor does not return to the same position each time when the wind becomes still, and that it is held away from the unstressed position a distance of the order of 0.5 in. The explanation of this phenomenon lies in the masonry envelop, which is of enormous stiffness against lateral forces, and has little strength. If it carried its own weight it would have great resistance to horizontal shear. Since it is supported at each story, the top layers of masonry in each story slide under horizontal shears, and the frictional resistance to sliding back prevents the building from returning to a zero position and keeps a small stress in the steel frame after the wind subsides. Except for this yielding resistance of the masonry the maximum deflections would be somewhat greater. In some of the top stories the masonry may carry nearly all of the wind shear, whereas in the lower stories it carries only a small part. In the lower stories the resistance of the masonry will not increase much as the wind force and the deflection increase. It is for this reason that structural engineers should neglect the masonry resistance in designing for wind loads. In the vibration of buildings, where the shears are small, the masonry plays an important part in determining the period and amplitude. The structural designer probably cannot do much to affect the vibration. The window area, type of masonry, and the character of the joint under the spandrels are more influential in respect to vibration than the stiffness of the structural frame.

A total deflection normal to the long axis of only 1.75 in. seems very small, but this was under a maximum velocity of only 50 miles per hr. Under 100 miles per hr the force would be four times as large, and many masonry joints, heretofore unstirred, would open and increase the deflection.

VICTOR R. BERGMAN,[47] Assoc. M. Am. Soc. C. E. (by letter).[47a]—In Part II, under "Introduction," Professor Rathbun states that "Rigidity is of considerable importance inasmuch as it affects the popularity of a building with tenants." This aspect of the problem is naturally uncertain and debatable. However, a structure that is too flexible may show a definite economic loss to the owner because of increased maintenance costs directly ascribable to lack of rigidity.

The writer has in mind an office building with a tower extending upwards beyond the 500-ft level above the street. After every heavy wind storm considerable plaster cracking is reported. For example, on Sunday, January 22, 1939, the wind velocity rose to 56 miles per hr; the next day no less than sixty-one complaints concerning cracked plaster deluged the maintenance department of the building! As might be expected, most of the cracks occur in the lower stories of the tower. They appear in the plastered gypsum block partitions, usually horizontally at the ceilings, or vertically at the ends. Apparently the cracking does not occur in the plaster on the brick panel walls, the "skeleton walls" of the building.

That this building sways during a heavy windstorm is patent even to a casually observant tenant of any of the uppermost floors. Some of the electric-

[47] Estimator and Engr., Godwin Constr. Co., New York, N. Y.

[47a] Received by the Secretary February 6, 1939.

light fixtures are suspended from the ceilings by means of tubes which have a universal joint arrangement at the upper ends. At times, their movement is quite perceptible.

During a heavy gale in 1938, two young women were permitted to leave their work to go home, following their pleas that they were suffering from "seasickness." Strangely enough, the tenants, with the foregoing exception, have not complained about the swaying of the tower. Indeed they seem to expect it and to look for it. The maintenance department of the building has even received telephone calls from the tower tenants concerning the swaying of suspended objects. "Do come up," they sometimes say, "it's very interesting."

Evidently the "scientific" articles on the subject, as "ladled out" by the Sunday supplements, have had a reassuring effect.

Corrections for *Transactions:* Line 4 below Fig. 2 should read "partitions except those surrounding the elevators and utility rooms. Fig. 4 is a typical"; on page 1361, next to the last line, change "Column (15)" to "Column (14)"; and, in Footnote 8, "Record" to "Forum" and "May, 1938" to "May, 1931."

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DISCUSSIONS

# PRINCIPLES APPLYING TO HIGHWAY ROAD-BEDS

## Discussion

By E. Neil W. Lane, Jun. Am. Soc. C. E.

E. Neil W. Lane,[34] Jun. Am. Soc. C. E. (by letter).[34a]—Settlement of a stabilized road-bed will occur as erosion proceeds and as the road-bed approaches the natural condition of top soil. The rate and amount of settlement will naturally vary as the intensity and frequency of the erosional processes.

Since all roadway economics is based on the economical life of the wearing surface, it is at once apparent that the most desirable road-bed is one that will serve its useful life over a period equal to the economical life of the wearing surface. This implies, of course, that it is quite as necessary to give the same careful attention to the design of the road-bed as it is to the layout of the surface course. Little information is available, it seems, as to the useful life of different stabilized road-beds. The effect on the useful life of the various weather factors, the type of surface protection (as under paving or forming its own wearing surface), the location of the road-bed (as to slopes and depths), and the amount and types of loads have been scarcely questioned as far as accurate detailed study is concerned. Soil stabilization is a young science, and there is naturally much confusion in classifying and interpreting the results of such existing examples and studies as have been made and in setting proper questions for the basis of future studies. The latter are of utmost importance at this time, especially since the most valuable tests are the results of experience under scientific conditions.

In this paper, Mr. Mullis has raised several open questions and has collected and interpreted a great mass of fundamental data. He is to be commended for his painstaking efforts and for the general and basic perspective from which he has viewed the subject. The use of available materials and economical design seem to be underlying key-notes of the paper.

Note.—The paper by Ira B. Mullis, Esq., was published in September, 1938, *Proceedings*. Discussion on this paper has appeared in *Proceedings*, as follows: February, 1939, by Messrs. A. F. Greaves-Walker, H. Z. Schofield, W. H. Campen, Bernard E. Gray, W. J. Turnbull, and Bert Myers.

[34] U. S. Dept. of Interior, Omaha, Neb.

[34a] Received by the Secretary January 9, 1939.

Many state specifications require the control of moisture and pore space but make little or no provision for placing the soil according to its ability to perform a service. Quite often they require that all the embankment be compacted to within 90% of the optimum compaction as shown by the Proctor density test. In doing this, it is generally found desirable to keep the moisture content slightly below (rather than slightly above) the optimum moisture as determined by the Proctor method, this being due to the fact that the usual construction equipment works more efficiently under these conditions. It would be better, the writer believes, to specify that the main body of fills be compacted to a stated lower density limit or such higher density as may be obtained by compacting the fill with a standard procedure and using a moisture content equivalent to 80% of the lower plastic limit, as shown by Atterburg's tests. Atterburg's tests are simple and thoroughly reliable, as have been proved. Moreover, it is believed that they are more adaptable to rapid field use in unskilled hands and with a minimum of equipment. Atterburg's tests do not allow as much variation of method and results as does the Proctor method.

The use of a sheepsfoot roller is obviously better than the smooth-wheeled roller for use on the major portion of a fill, although the latter is necessary at and near the surface. The specifying of the two types of rollers for each project is a nicety to be developed by the future.

As a result of present-day experiences with field conditions and specifications, it would seem desirable to propose certain changes in the control of the construction of stabilized road-beds for the sake of a cheaper construction cost for an equivalent road-bed as far as useful life is concerned. These propositions are not as economically important for shallow cuts and fills as they are for heavy earthwork and side-hill construction.

It has been noticed that many road-bed troubles occur at transition points between a cut and the adjacent fill. This is undoubtedly caused by the rapid change in thickness and density of the road-bed at such points. In the case of rigid pavements an extreme condition can be shown for the purpose of illustration, although the results are noticeable in the non-rigid surfaces with a more deteriorating effect.

Assuming that the span between suitable bearing in cut and fill is the practical equivalent of two slab lengths of pavement and that the joint between these slabs is approximately over the thinnest portion of the stabilized road-bed, it is easily seen how undesirable variations in the density of this thin portion of the road-bed can cause serious surface misalignment in the case of rigid paving and a gradual disruption in the case of the non-rigid surfaces.

To overcome this difficulty it would seem desirable to excavate the ends of a cut first to such a depth as is necessary to obtain a natural soil with suitable density, or to excavate to some depth as would seem judicious, and replace this soil with stabilized material. The material excavated at the ends of the cut should be hauled to the deepest portion of the fill, and replaced by soil from the middle of the cut. This eliminates duplicate hauling but necessitates an additional handling charge which the writer believes would be more than offset by a decrease in maintenance and replacement expenses.

The variation of density with the depth of fill is a subject which Mr. Mullis has treated rather lightly in view of its importance, although there is a dearth of information on this subject. It is reasonable that the density at any depth of a stabilized road-bed should vary as the load above it and inversely as the width of the base. Since the bearing width increases at a greater rate than the depth, appreciable savings in road-bed construction could be obtained by using a less dense stabilization in the wider portions of the fill which make the larger portion of the embankment. Due to the desire to distribute concentrated wheel-loads and cut down the action of capillary water adjacent to wearing surfaces, it is naturally desirable to require the most rigid control over stabilization in the first few feet below the surface course. This region, being the most exposed and least confined portion of the road-bed, should be the most dense. However, the depth of this layer of maximum density need not be the same for all conditions. With a protective covering of concrete paving one could quite reasonably expect a satisfactory performance with a high density layer of, say, a 2-ft minimum thickness whereas under a bituminous mat it would obviously be much better to have a minimum depth of 4 ft or 5 ft.

The use of a thin layer of granular stabilization (2 in. to 6 in.) between concrete paving and the high density layer is being studied with a view of decreasing the action of moisture on the bottom surface of the slab. It is expected that the moisture coming through the joints, cracks, and at the sides of slab will spread over a wide area on reaching the granular layer, thus producing a light and uniform percolation downward through the road-bed. In this way the layer will act as an insulator. A secondary effect will be to decrease the bond between the slab surface and road-bed, thus lowering internal stresses in the paving due to temperature changes and creep from hillside location.

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DISCUSSIONS

# ENERGY MASS DIAGRAMS FOR POWER STUDIES

## Discussion

By Lyle A. Whitsit, M. Am. Soc. C. E.

Lyle A. Whitsit,[5] M. Am. Soc. C. E. (by letter).[5a]—That a mass curve of energy may be used to analyze a power supply in a manner similar to the use of a mass curve of flow to analyze a water supply, has been demonstrated in this paper. When the water supply is first determined by a mass curve, the result is then combined with the net head for power determination. If the power-generating system consists of a series of reservoirs and power houses on one stream and its tributaries, the use of water is in a concatenated sequence. The water availabilities at one plant depend upon the use of water from the reservoirs and in the plants up stream. The analysis by means of a mass curve of stream flow becomes complicated and cumbersome to the $n$th degree. As a result, the writer, like the author, proposed[6] the use of a mass curve of energy in 1913. However, even the use of the mass curve of energy is an unsatisfactory method.

In the investigations and reports prepared since his earlier work, the writer has never found the use of the mass curve of energy advisable because of its complicated character and the precautions, adjustments, and corrections that are required. Graphs accompanying reports must necessarily be easily read, and their determination must be understandable to executives and other readers of a report. For this reason the writer believes that the use of duration curves of flow, or hydrographs, in combination with duration curves of load, are preferable.

Analysis by several duration curves or power graphs can be incorporated into one diagram. It is a method used in other analysis, such as the superposition of the hourly output of two or more generation stations, to produce a combined daily load curve of a power system. Such graphs can be made to show clearly the source of stored energy for later use, such as night storage for day-time or week-time peak-load use in the analysis of resources possessing

Note.—The paper by John W. Hackney, Jun. Am. Soc. C. E., was published in October, 1938, *Proceedings*. Discussion on this paper has appeared in *Proceedings*, as follows: January, 1939, by Edgar E. Foster, Assoc. M. Am. Soc. C. E.

[5] Cons. Hydr. Engr., Ebasco Services, Inc., International Div., New York, N. Y.

[5a] Received by the Secretary December 15, 1938.

[6] *Engineering News*, September 11, 1913, p. 496.

pondage; or such as storage during high-water period for any rate of use during low-water periods.

At present (1939) there are very few power companies with generating systems made up entirely of hydro prime movers. Practically all have a complement of steam or other thermal generated capacity. In fact, it is very seldom that a hydro-electric project by itself can compete with a steam-electric project by itself. The proper position of hydro capacity, especially when it is equipped with storage regulation, is primarily one furnishing capacity, large amount of kilowatts for a small consumption of energy. Energy can be produced usually at much less cost by steam.

The economical arrangement results in the bulk of energy being produced from thermal plants because of lower cost; but hydro plants with pondage and storage can compete effectively in the production of capacity, that is, kilowatts of power in the peak of the load at a small energy consumption. Quite often an additional installation may be provided in such hydro plants to serve as the standby capacity for the system. When this portion of the installation is serving as standby for the other hydro capacity it is doing so at no extra consumption of energy, but when serving as standby for a base thermal capacity its energy use is at the detriment of the hydro peak carrying capacity. Perhaps the economical arrangement is to split the standby provision into hydro for the hydro load capacity and steam for the steam portion.

For this reason other forms of graph analysis, such as the duration curve or hydrograph, are more convenient and more commonly used in preliminary and final steps of the determination of a "load-carrying capacity" of a proposed project. In the determination of the load-carrying capacity, the plant output with unregulated water supply up stream from reservoirs is allocated to base load. The plant output of an unregulated water supply close to an up-stream storage plant is essentially a part of the storage plant and is allocated to peak load. The output of a plant with an unregulated water supply below a storage plant but with its own large tributary drainage area is probably allocated to base load.

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DISCUSSIONS

# BEAM CONSTANTS FOR CONTINUOUS TRUSSES AND BEAMS

## Discussion

By Messrs. George W. Lamb, and A. A. Eremin

George W. Lamb,[11] Assoc. M. Am. Soc. C. E. (by letter).[11a]—In making possible the computation of stresses in trusses by practically all the methods that are used for beams, this paper is especially commendable. Thus, it furthers the use of indeterminate structures. Although the writer does not agree that the method of analysis presented is more accurate or more convenient to use than any of several other methods, or combination of methods, the procedure recommended does have some distinct advantages on certain types of trussed structures, especially trussed frames, and similar structures that are commonly analyzed by use of moment distribution or slope deflection.

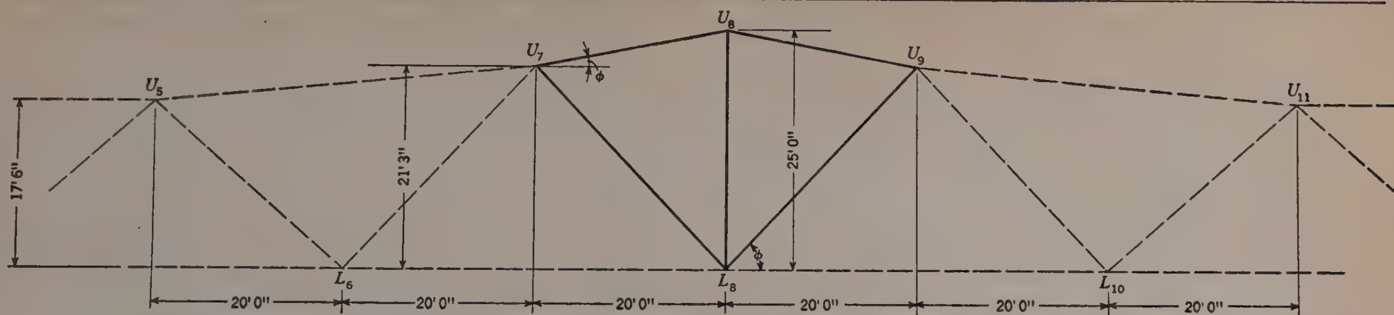
In all methods of stress analysis and design, considerable value should be placed on the capability of any step in the analysis to lend itself to the calculation of other properties of the structure. In the method presented by Mr. Epps, this is true to some degree. The influence of the change in stress in the member over the pier on any other member in one span of the truss can readily be calculated. However, there are several other properties and conditions on continuous trusses that are always necessary to compute or investigate, to which the method does not adapt itself so readily. Some of the conditions and properties that this method does not handle easily are as follows: (1) Influence lines for all the members of the truss; (2) the effect of settlement of supports and the data necessary for adjustment of reactions; (3) the calculation of dead-load deflections; and (4) it is customary to camber continuous trusses by adding to, or subtracting from, the normal length of the chords and webs, the deformation caused by the stress produced by the load for which the structure is cambered. These deformations are small and are usually given on plans to the nearest $\frac{1}{16}$ in. The differences between the fabricated dimension to the nearest $\frac{1}{16}$ in. and the deformation caused by the load are small, but the

Note.—This paper by George L. Epps, Jun. Am. Soc. C. E., was published in October, 1938, *Proceedings*. Discussion on this paper has appeared in *Proceedings*, as follows: February, 1939, by C. W. Deans, Esq.

[11] Asst. Bridge Engr., Kansas State Highway Comm., Topeka, Kans.

[11a] Received by the Secretary December 16, 1938.

TABLE 3.—SUGGESTION FOR TABULATING COMPUTATIONS




| CONSTANTS | | | | | | STRESSES | | | | REACTIONS | | | | | | DEAD-LOAD DEFLECTIONS | | | | CAMBER | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| Member | Area, $A$, in square inches | Length, $L$, in inches | Sec $\phi$ | Height of panel, in feet | $\frac{L \sec \phi}{A E h}$* | Unit load at Point $L_8$ | Unit load at Point $L_{8'}$ | Dead-load stress, in kips† | Camber stress, in kips† | Joint | Elastic weight; unit load | Elastic shear | Elastic moment | Ordinate of Point $L_8$ | Ordinate of Point $L_{8'}$ | Elastic weights; dead load | Elastic shear | Elastic moment | Dead-load deflection, in inches | Elastic weight camber | Elastic shear | Elastic moment | Camber, in inches |
| (1) | (2) | (3) | (4) | (5) | (6) | (7) | (8) | (9) | (10) | (11) | (12) | (13) | (14) | (15) | (16) | (17) | (18) | (19) | (20) | (21) | (22) | (23) | (24) |
| $U_7 L_6$ | 27.04 | 350.2 | 1.46 | 25 | 0.756 | −0.190 | −0.084 | −185 | −290 | $L_6$ | +4.68 | +34.6 | 302 | +0.927 | +0.163 | + 72.1 | −563 | 825 | 0.550 | − 71.9 | +631 | 1 095 | 0.730 |
| $U_7 U_8$ | 28.21 | 244.2 | 1.02 | 25 | 0.352 | −4.51 | −2.00 | +298 | +433 | $U_7$ | +6.53 | +28.1 | 336 | +0.984 | +0.071 | −301 | −262 | 262 | 0.175 | −265 | +366 | 464 | 0.309 |
| $U_8 U_9$ | 28.21 | 244.2 | 1.02 | 25 | 0.352 | −4.51 | −2.00 | +298 | +433 | $L_8$ | +3.14 | +25.0 | 364 | +1.00 | +0 | −535 | +293 | 0 | 0 | −837 | −470 | 97 | 0.064 |
| $U_9 L_8$ | 38.23 | 300.0 | .... | 20 | 0.392 | +1.66 | +0.738 | −110 | −239 | $U_9$ | +6.67 | +18.3 | 389 | +0.992 | −0.061 | −311 | +604 | 293 | 0.195 | −265 | −735 | 567 | 0.378 |
| $L_8 U_9$ | 27.04 | 350.2 | 1.46 | 25 | 0.756 | +0.718 | −0.930 | −210 | −290 | $L_{10}$ | +5.79 | | 407 | +0.944 | −0.101 | + 79.7 | | 897 | 0.598 | + 56.6 | | 1 301 | 0.868 |

* $E = 1$. † 1 kip = 1 "kilo-pound" or 1 000 lb.

total effect of these small differences on the total truss is sufficient in some cases to cause some trouble in erection closure, and the effect should be taken care of by raising or lowering the supports or by adjusting the length of some member with a high elastic constant. Therefore, in some cases it is necessary to calculate camber in the truss with the deformation in the members different from that which the loads on the truss would indicate.

The difficulty of handling the first and second conditions can be blamed directly on the method. That is, in the first condition the influence line for the member over the support is obtained easily, but in order to obtain the influence line for a member in the truss between supports, it is necessary to have influence lines for shears and moments at each support. This involves considerable work. The second condition, settlement of supports, is also rather tedious to calculate using the constants obtained, as it involves the elastic properties of all the spans in the unit.

The third and fourth conditions, the writer believes, can be handled more easily by using a different method of calculating the elastic weights and another form for tabulating them. The writer prefers to use the method of computing angle changes in obtaining the elastic weights. Table 3 shows the value of using one set of calculations for several different properties of the truss. The diagram is one panel of a continuous truss over the reaction. The elastic shear and moments are carried over from the dotted part of the truss. It will be noted that the values given in Column (6) are used for obtaining the influence-line ordinates for reactions, the dead-load deflections, and the camber ordinates. The values shown under Column (17), with small modifications, can be used in the calculation of secondary stresses for dead load. The values given in Columns (13), (14), (18), (19), (22), and (23) are relative, and are considerably larger than the true values. This was done to obtain numbers that are easier to handle. To obtain the true deflection in inches, the elastic moments must be multiplied by $\frac{12 \times 1\,000 \times 20 \times 12}{144 \times 30\,000\,000} = \frac{1}{1\,500}$. The elastic weight given to Point $L_8$, Column (12), is obtained as follows: $-(+\,0.190 \times 756 - 4.51 \times 0.352 - 4.51 \times 0.352 - 1.66 \times 3.92 + 0.718 \times 0.756) = 3.14$ with due attention being paid to the sign of the stress and whether the member is a web or a chord member, the minus sign in front of the parenthesis shows that the angle at $L_8$ decreases under this condition of loading. The foregoing example is part of the calculation used in designing a three-span continuous truss by the method of redundant reactions; it shows a different method of tabulating values so that they can be used in calculating various properties. It is also different inasmuch as the distances to the intersection of the members do not have to be calculated, and the dimensions and stresses within the panel are sufficient to obtain the elastic weight at any point.

The accuracy of the method of redundant reactions, as presented by Mr. Pierce,[2] is questioned in this paper on the basis of the fact that in order to be accurate the large numbers must be extremely exact because of the small differences to be obtained. In the method of redundant reactions this objection

[2] "Simplified Solutions of Influence Lines for the Reactions of Continuous Beams," by V. L. Pierce, *Civil Engineering*, December, 1935, p. 798.

to great exactness on this basis can be avoided as follows: Instead of taking the difference between two large deflections, the deflection of the structure is obtained at all points with a unit deflection at the support for which the influence line for reaction is desired. Consider, for example, a four-span continuous beam having a constant moment of inertia and having spans of 65 ft, 80 ft, 80 ft, and 65 ft. In order to obtain deflections easily, the total length of the unit is changed to a length of unity. Let $E = 1$; and $I = 1$. The beam shown in Fig. 9 is the result. To obtain the equation for the influence line for the reaction at Point ($a$), Fig. 9, a unit load will be placed at Point ($a$) and the equation for the influence line will be written Determine the reactions at Points $A$, $b$, $c$, and $B$ with a unit load at Point ($a$)

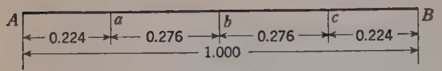



FIG. 9

$y_{bb} = \frac{1}{48} = 0.0208$; $y_{bc} = 0.0131 = y_{ba} = y_{ab} = y_{cb}$; $y_{cc} = 0.0101$; $y_{ac} = 0.0075 = y_{ca}$; for example, $y_{bc}$ = the deflection at $b$ with a unit load at $c$, etc. Substituting:

$$0.0208\, R_b + 0.0131\, R_c + 0.0131 = 0 \qquad (7a)$$

and

$$0.0131\, R_b + 0.0101\, R_c + 0.0075 = 0 \qquad (7b)$$

Solving Equations (7) simultaneously: $R_b = -0.851$; $R_c = +0.356$; $R_A = -0.430$; and $R_B = -0.075$. Applying these reactions to the beam a moment diagram is obtained as shown in Fig. 10.

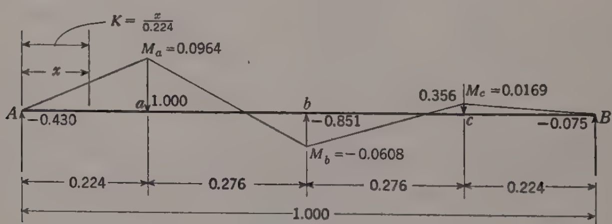



FIG. 10

From elastic shear, the slopes at the supports are as follows: $\phi_A = -0.0109$; $\phi_a = -0.00012$; $\phi_b = 0.00481$; $\phi_c = -0.00124$; and $\phi_B = +0.00062$. Use these slopes and write the equation of the elastic line in Span $A$–$a$ as follows:

$$\frac{d^2y}{dx^2} = 0.430\, x \qquad (8a)$$

$$\frac{dy}{dx} = 0.215\, x^2 - 0.0109 \qquad (8b)$$

and

$$y = 0.0717\, x^3 - 0.0109\, x \qquad (8c)$$

When $x = 0.224$, $y = -0.00164$.

To obtain a unit deflection at Point ($a$) and to make Equation (8$c$) apply to any four-span continuous unit with the same ratios of span lengths, multiply this equation by $-\frac{1}{0.00164}$ and substitute, letting $x = 0.224\ K$, and letting $O_{A-a}$ = ordinate for influence line for the reaction at Point ($a$) with a load in Span $A$–$A$: $O_{A-a} = -\frac{1}{0.00164}(0.0717 \times \overline{0.224}^3\ K^3 - 0.0109 \times 0.224\ K)$; or

$$O_{A-a} = 1.49\ K - 0.49\ K^3 \ldots\ldots\ldots\ldots\ldots\ldots (9)$$

and the area under this curve for uniform load equals 0.622. Using the foregoing reactions and slopes, the equation for the reaction at Point ($a$) with the load anywhere in the unit can be found; and a considerable portion of the foregoing information can be used in obtaining the influence lines for the other reactions. These calculations can be made with a slide-rule to the accuracy required for designing any structure. When applied generally, the foregoing procedure can be used for finding the influence line for reaction on trusses and beams of varying moments of inertia. Although it appears rather cumbersome this general method, when used by any one familiar with it, is as accurate and requires as little work for the complete analysis of a structure as any other that may be used, and also offers several checks on arithmetical errors. The complete analysis includes all shears, moments, deflections, camber, settlement of support, investigations, etc.

Any sound method of design, even when used in conjunction with relatively rough calculation, is far more accurate than the assumptions of dead load and live load applied to the structure. For instance, reinforced concrete is usually assumed to weigh 150 lb per cu ft. If this varies even 1.5 lb per cu ft, the dead load of the concrete would be inaccurate 1 per cent. Even structural steel will vary in weight from the theoretical, and live loads on most structures can and do vary widely from the assumed live load. This is not intended to imply that the calculations can be as erroneous as the assumption for load, but it is intended to show that the method used in the analysis is perhaps the most accurate part of the design even when handicapped with relatively rough calculations.

A. A. Eremin,[12] Assoc. M. Am. Soc. C. E. (by letter).[12a]—A method of analyzing stresses in continuous trusses by computing bending moments at the supports is presented in this paper. Mr. Epps has cited various advantages of the method over the conventional method in which the reaction forces must be computed. These advantages are even more numerous than those indicated. As long ago as 1930 Prof. G. G. Krivoshein showed that, by computing bending moments at the supports of continuous trusses, the results will be more accurate than by computing the redundant forces at the supports.[13] Professor Krivoshein developed various equations for computing bending moments at the supports.

[12] Associate Bridge Engr., Bridge Dept., Div. of Highways. State Dept. of Public Works, Sacramento, Calif.

[12a] Received by the Secretary December 27, 1938.

[13] "Simplified Calculations of Statically Indeterminate Bridges," by G. G. Krivoshein, Prague, Czechoslovakia.

The elastic properties of continuous trusses may be expressed by:

$$\frac{I}{I_0} = \left(\frac{d}{d_0}\right)^3 \dots\dots\dots\dots\dots\dots\dots\dots (10)$$

in which, $I$ and $d$ are moment of inertia and depth of truss at any section, respectively; and $I_0$ and $d_0$ are some arbitrarily selected moment of inertia and depth of truss, respectively.

In some trusses Professor Krivoshein stated that satisfactory results may be obtained with the exponent in Equation (10) equal to 2.5 instead of 3. Obviously, Equation (10) is exactly the same as that generally used in the analysis of stresses in continuous beams of solid cross-section and constant width. It is interesting to note that this formula may also be used in computing the constants of continuous trusses.

Mr. Epps computes the elastic weights of continuous trusses by means of the radii. However, a more direct computation may often be made by means of formulas previously cited by the writer,[14] in which the elastic weights are computed directly from the angular relation between truss members at each joint. Furthermore, the terms in the writer's equation[14] may also be used for analyzing secondary stresses in the trusses, thus saving time when additional computations for the complete analysis of stresses is required.

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[14] *Transactions*, Am. Soc. C. E., Vol. 102 (1937), p. 209.

AMERICAN SOCIETY OF CIVIL ENGINEERS

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DISCUSSIONS

# SOLUTION OF EQUATIONS IN STRUCTURAL ANALYSIS BY CONVERGING INCREMENTS

## Discussion

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BY A. A. EREMIN, ASSOC. M. AM. SOC. C. E.

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A. A. EREMIN,[11] Assoc. M. Am. Soc. C. E. (by letter).[11a]—The solution of equations with more than three unknowns is a complicated task even with the

| | B | | C | | D | | E |
|---|---|---|---|---|---|---|---|
| Stiffness Factors | 33<br>10.5 | | 28.8<br>8.5 \| 6 | | 44.8<br>14 \| 8.4 | | 42<br>7 |
| Carry-Over Factors | | 0.255 →<br>← 0.347 | | 0.487 →<br>← 0.134 | | 0.166 →<br>← 0.20 | |
| Fixed-End Moments | −2 385 | | −2 045 | | −1 412 | | −1 575 |
| 1st Distribution | −2 385<br>−524 | →<br>← | −607<br>−1 438<br>−96 | →<br>← | −698<br>−714<br>−293 | →<br>← | −112<br>−1 463 |
| 2nd Distribution | 524<br>−13 | →<br>← | 133<br>−37<br>42 | →<br>← | −18<br>311<br>−10 | →<br>← | 49<br>−49 |
| 3rd Distribution | 13<br>−16 | →<br>← | 3<br>−45<br>4 | →<br>← | −22<br>32<br>−1 | →<br>← | 5<br>−5 |
| 4th Distribution | 16<br>−3 | →<br>← | 4<br>−8 | → | −4<br>5<br>0 | →<br>← | 1<br>−1 |
| Sums | −1 832 | | −1 528 | | −366 | | −1 518 |
| Moments | −55.3 | | −53.1 | | −8.2 | | −36.1 |

FIG. 8.—DISTRIBUTION OF BENDING MOMENTS

aid of a computing machine. It is gratifying, therefore, to encounter a simplified method, such as that presented by Mr. Dell, based on the principle of

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NOTE.—The paper by George H. Dell, Assoc. M. Am. Soc. C. E., was published in October, 1938, *Proceedings*. Discussion on this paper has appeared in *Proceedings*, as follows: February, 1939, by L. E. Grinter, M. Am. Soc. C. E.

[11] Associate Bridge Engr., Bridge Dept., Div. of Highways, State Dept. of Public Works, Sacramento, Calif.

[11a] Received by the Secretary January 11, 1939.

converging approximations. The computations may be still further simplified as demonstrated in Fig. 8, as applied to Equations ($B$) to ($E$) of the paper.

The stiffening factors in Fig. 8 are written in the same order as they appear in the equations. The carry-over factors were computed with stiffening factors at the joints and have logical directions. The distribution of fixed-end moments is made in a similar manner to that of the paper. The order of computations shown in Fig. 8 is easy to memorize and is in a convenient form, designed for greater assurance against errors of computation.

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DISCUSSIONS

# RECONSTRUCTION OF A PIER IN BOSTON, MASSACHUSETTS, HARBOR

## Discussion

BY JOHN N. FERGUSON, M. AM. SOC. C. E.

JOHN N. FERGUSON,[4] M. AM. SOC. C. E. (by letter).[4a]—The reconstruction of the platforms at Commonwealth Pier No. 5, Boston Harbor, was due to the action of marine borers on wooden pile foundations which made it necessary for a more permanent type of foundation to be provided. The unique and unusual features of the type of construction adopted are ably and concisely described in the paper by Professor Spofford. However, there are a few other items which may be of interest.

In addition to the creosoted piles used in the new foundations for the platforms at the head-house, creosoted timber piles and timbers for bracing were used for fenders along the outside edge of all platforms, and as fender clumps at the corners of the pier.

The specifications for creosoting read, in part, as follows:

> "All new piles driven in the fender system and platforms shall be southern yellow pine * * * . The piles shall be treated by the full cell process with 16 lb per cu ft of Grade 1 Creosote Oil. All timber below high-water line shall be treated by the full cell process with 12 lb of Grade 1 Creosote Oil * * * . The fender caps and the top planking of the corners may be treated with preservative salts if the Contractor so desires * * * ."

The fender system along the edge of the platforms consists of two plumb piles and a spur pile in sets spaced 10 ft on centers. The plumb piles are placed one behind the other, and the spur pile is bolted to the inner plumb pile just below a wale which is about 5.5 ft above low water. The top of the inner plumb pile is fitted with girder caps which are bolted to the platforms at intervals. The outer plumb pile is bolted to the inner plumb pile through a continuous longitudinal wale which extends between the plumb piles about 2 ft

NOTE.—The paper by Charles M. Spofford, M. Am. Soc. C. E., was published in October, 1938, *Proceedings*. Discussion on this paper has appeared in *Proceedings*, as follows: January, 1939, by William G. Atwood, M. Am. Soc. C. E.

[4] Dist. Waterways Engr., Dept. of Public Works of Massachusetts, Boston, Mass.

[4a] Received by the Secretary December 21, 1938.

above low water. By this arrangement full flexibility of the upper 16 ft of the outer plumb pile is utilized.

The structural frame for the platform on the piles abreast of the head-house consists of reinforced concrete girders and beams instead of the silicon steel frame work that was used abreast of the freight sheds. To fasten the girders and beams to the piles, an inverted U-bolt was placed in the head of each timber pile.

The placing of the additional load of light-weight concrete on the pile clumps supporting the columns of the head-house has presented no apparent settlement, as might be evidenced by cracks in the plastering and artificial stone of the face of the building.

To protect the platform slab, a part of which was placed during cold weather, the contractor provided some trackmen's railroad hand-cars on which was built a frame of wood extending over two bays of the platform. This frame was covered with canvas, and heat to prevent freezing was supplied from the boilers of the contractors' floating equipment. As the work progressed the weather protection was moved along to a new section on the standard-gage track which extended the full length of the side platforms.

The concrete used in the platforms was all transit mix of the following proportion by volume: 1 part cement, 2 parts sand, 3.5 parts coarse aggregate, and 5.75 gallons of water.

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DISCUSSIONS

# TRANSPORTATION DEVELOPMENTS IN THE UNITED STATES

## Discussion

By Messrs. L. Alfred Jenny, William J. Cunningham, S. R. Truesdell, David A. Molitor, and C. A. Hoglund

L. Alfred Jenny,[25] M. Am. Soc. C. E. (by letter).[25a]—The railroad problem is a very live issue at present. The writer is in general agreement with Mr. Lavis' analysis of the situation and his conclusion that the railroads are, and will continue to be, the major transportation medium. Some national research bureau would be very valuable. Much could be accomplished by the use of lighter and more modern equipment. Certainly something must be done to save the railroad industry. The writer does not agree with some of Mr. Lavis' conclusions, however, or the solution he offers. He agrees with the general diagnosis, but disagrees on the cure.

*Are Many of Our Railroads Over-capitalized?*—Mr. Lavis maintains that the railroads of the United States are not over-capitalized, and he thinks it is not necessary to put many of them "through the wringer." There are many schools of thought on this subject, and it all depends upon the standard by which the capital structure is judged. If it is judged on the basis of valuation for certain periods, then many capital structures compare favorably; but it is not fair to stop there without also taking into consideration the past, present, and future potential earning power of the property.

Abundant examples are available of industrial plants built to meet increasing demands for their product, which could not even be duplicated to-day for the price paid originally, but which now have very little value, not from a physical point of view, but because they are being operated only at a fraction of their possible capacity, and because there appears little chance of operating

Note.—This paper by Fred Lavis, M. Am. Soc. C. E., was presented at the Annual Convention, Salt Lake City, Utah, on July 20, 1938, as part of the Symposium on Transportation, and was published in November, 1938, *Proceedings*. Discussion on this paper has appeared in *Proceedings*, as follows: February, 1939, by Messrs. Milton Harris, and Ralph Budd.

[25] Cons. Engr. (L. Alfred Jenny & Co.), New York, N. Y.

[25a] Received by the Secretary December 15, 1938.

fully again. These industrialists have long since taken their losses and have written off a very substantial portion of the original investment.

The argument may be raised that one cannot compare an industrial plant with a railroad; perhaps not. The railroad picture is even worse, because the industrialist can close a part or all of his plant if need be, whereas the railroad cannot, and, therefore, should have a financial structure that will enable it to remain solvent in depression periods.

Railroads were built to meet what looked like an ever-increasing flow of traffic and in a period when competition from other forms of transportation was negligible. Now, with increasing competition, many of them cannot possibly hope to operate to a capacity sufficient to compensate properly all classes of investors. Many of these railroads have not even been able to pay interest charges on income bonds during good years, when there was very little other competition, to say nothing of payments on the preferred or common stock. In fact, there have been many bankruptcies even during prosperous years. The common stock has received improper consideration in many instances—a very unfair procedure.

If a railroad is properly capitalized it should be able to pay a fair return on all classes of securities (even on the common stock) during prosperous times. If it cannot do that, it is difficult to understand how one may claim that it was not over-capitalized, or that it was not necessary to reduce its capital structure to such an extent as to permit a reasonable return for all classes of security holders in good times. The Interstate Commerce Commission has stated that, in general, the railroads are not over-capitalized. The writer believes that this statement was based on the valuations; yet the Commission wants to reduce fixed obligations drastically; in many instances it has held that the common stock had no value and has recommended that these securities be eliminated from consideration in reorganization proceedings. If that is not evidence of over-capitalization, one may ask, "What is?" Many railroads are, in fact, over-capitalized and many of them must be put "through the wringer" in order to effect a cure and not merely a palliative which perpetuates a diseased body and even adds new complications. One of the fundamental difficulties of the carriers is that so many of them have been continually adding new security issues to meet new requirements and, instead of following a balanced program of cancellation when they became due, have resorted to the policy of refunding. Although many of them had no alternative, it has made their financial position increasingly precarious.

*Added Federal Loans.*—Mr. Lavis is in favor of Government credit and the making of Government loans to the carriers and States (see heading, "The Solution: Government to Furnish Capital and Credit to Railways"), and "if the railroads are to avoid Government ownership and operation to which the President says he is opposed, there is little else that can be done." Here too there are many schools of thought; but one may ask seriously how it is expected to repay these loans, or even to pay interest on them, if no drastic measures are taken at the same time to revamp the entire industry and "put it on its feet." Under the circumstances, such loans will only add to the financial woes of the

carriers and help to force early Government ownership, the very effect Mr. Lavis hopes to avoid. It is probably true that the Government will have to extend credit to the railroads and make substantial loans to permit rehabilitation, but that should not be done without, at the same time, setting up proper machinery for bringing about a reconstitution of these properties and a sound national plan of co-ordination and correlation of all forms of transportation.

*Deferred Maintenance.*—Mr. Lavis states that the deferred maintenance at the end of 1937 was estimated to be approximately one billion dollars. The writer believes that it was more than that, based upon a study in which he compared the maintenance costs of the five years, 1932–1936, inclusive, with those of the seven years, 1925–1931, inclusive.

As the cost per equated ton-mile was deemed to offer the best relative comparison, that method was used. It is fully appreciated, however, that some labor-saving devices have been installed in recent years which did not exist during the late twenties, and that some processes have been developed for extending the lives of rails, ties, etc., and that these have somewhat reduced the cost of comparable maintenance. By making a direct comparison between, say, 1925–1929 and 1932–1936, the resultant value, therefore, would tend to show a slightly greater sum for deferred maintenance than actually existed. In order to be conservative, and in some way to compensate for this condition, the first two depression years, 1930 and 1931, were added to the base period. This introduced an equalization factor of about 10%, or, in other words, the resultant value, covering all maintenance expenditures including materials, etc., is about 10% less than it would be if the years 1925–1929 were used as the base period.

The result of that study, and on the basis outlined, is shown in Table 4.

TABLE 4.—STUDY OF DEFERRED MAINTENANCE, CLASS 1 RAILROADS

| MAINTENANCE OF: | (a) UNDER-MAINTENANCE, IN MILLS PER OPERATED TON-MILE | | | | (b) TOTAL FOR THE FIVE YEARS, 1932 TO 1936, INCLUSIVE, IN DOLLARS | | | |
|---|---|---|---|---|---|---|---|---|
| (Districts) | Eastern | Southern | Western | Total | Eastern | Southern | Western | Total |
| Way and structure | 0.42 | 0.33 | 0.36 | 0.380 | 283 400 000 | 114 300 000 | 191 800 000 | 589 500 000 |
| Equipment | 0.27 | 0.14 | 0.11 | 0.185 | 182 100 000 | 48 500 000 | 58 600 000 | 289 200 000 |
| Total | .... | .... | .... | ..... | 465 500 000 | 162 800 000 | 250 400 000 | 878 700 000 |

The total under-maintenance on Class 1 railroads, alone, for the five depression years, 1932–1936, has reached the staggering sum of nearly 900 million dollars and if Class 2 and 3 railroads, and switching and terminal companies are added, the total will be much greater. If 1937 and 1938 are added, it is believed that the total for deferred maintenance will be nearly 1½ billion dollars.

If the deferred maintenance was estimated to be about 1 billion dollars at the end of 1937, it would probably be about 1⅛ billion dollars to-day (1939) on that basis. Whether it was that value, or 1½ billion dollars, or some intermediate value, is no doubt a question of judgment. The important question is,

"How is it to be met?" No one can give the answer at this time. One thing is certain: It will represent a staggering sum, a "millstone around the necks" of these carriers which, for an indefinite period, will prevent them from enjoying even a relative economic freedom, so essential to the up-building and stabilization of that industry, and which will greatly retard the rehabilitation of these properties.

When it is considered that some of these railroads have maintained their properties along a higher standard than others, it must be obvious that some carriers will be found to be below the average standard of the district. Such a continued under-maintenance is certain to become dangerous if prompt measures are not taken to obtain an early remedy.

It must be obvious from this that maintenance costs in the future, at least for many years to come, will be more, instead of less, than in the past, and that if any operating savings are to be made they must be made in other ways than at the expense of proper and adequate maintenance.

*Labor.*—The United States has not been immune from the surges of varying philosophies going in one direction or another, and there has been a considerable change in attitude, particularly with regard to labor problems. Some call it the emancipation of labor; others call it by a less euphemistic name. Whatever it may be called, it is something to conjure with. It cannot be disregarded. The proper thing to do would seem to be to recognize its existence and assist in directing it along sound, equitable, and democratic lines.

In reviewing the railroad situation the labor problem must therefore be considered seriously, particularly in its effect upon possible future net earnings. During low traffic periods in the past, the railroads were able to reduce wages and thus correspondingly reduce operating costs, resulting in a more uniform ratio between income and outgo. All statistics, including maintenance, therefore, bear some relation to the prosperity of the carrier. It seems doubtful if such a procedure can be followed in the future.

The 1938 decision in regard to the reduction of 15% in railway wages is a "shining example" of this changed philosophy. No one can deny that this decision will have a tremendous and far-reaching effect upon the future of the railroad industry. Only a few years ago, and under similar economic duress, railway labor would have been almost certain to lose, or at best effect a compromise. Failure to recognize this condition can only result in disappointment later.

The railroads had estimated a net saving of about 250 million dollars annually with the 15% wage reduction they had asked for, and they stated frankly that, if such a reduction were not made, serious consequences would result. No reduction has been made, and there appears to be little likelihood that any will be made, of sufficient importance to have any material effect upon the net operating returns of the carriers.

Railroads must be prepared, therefore, to effect important savings in other ways because the carriers cannot continue to function properly under the existing heavy burden of expenses. Some people have suggested a profit-sharing plan for labor as a possible solution. It would have the effect of providing an equalizing factor, fluctuating with the prosperity of the carrier. It may well

be that a solution along these lines could be effected which would establish a minimum fixed return for both capital and labor, and permit labor to share with the stockholder the fruits of a more abundant day.

*The Problem.*—The problem that confronts the nation is much broader and much more deeply rooted than what is commonly referred to as the "railroad problem." In order to grasp this fact fully it is necessary to review some of the underlying factors briefly. With few exceptions the railroads were built with private capital. They enjoyed a monopoly on transportation and attracted great sums of money for new construction to meet an expanding trade. Although the inland waterways, developed at public expense, had long predated the railroads, they offered no formidable competition, particularly with reference to the growing transcontinental traffic. Then the Government built the Panama Canal, at public expense, and this project detracted great tonnages from the transcontinental railroads of the United States. Not only is that tonnage lost, but the diversion is increasing.

New ports were established, at public expense, along the Gulf of Mexico, and much of the rail tonnage, particularly between Texas and Louisiana and the Eastern seaboard, is now (1939) handled by coastwise shipping, in which field competition is increasing.

Pipe lines were then built, taking the oil away from the railroads. In addition to losing the oil business this development has had the effect of substituting oil for coal in many fields. As coal is one of the most profitable carrier commodities, this trend has reduced railroad revenues seriously.

Large-scale highway developments were made necessary by the introduction of the automobile. They were built at public expense, and passenger and freight-carrying vehicles, both public and private, were permitted to operate over them in competition with the railroads. This agency has proved to be by far the most serious competitor of the carriers. In addition, it has made necessary very large railroad expenditures for grade-crossing eliminations, thus not only reducing carrier income, but increasing outgo.

Finally, there has been a great development in the air industry. Most of the airports were built by municipalities, again at public expense, and since no traffic arteries had to be built, with perhaps the exception of beacons, the airplane too began to compete over public facilities.

The truck and the bus are, in fact, making such inroads upon the carriers, in so far as traffic within a radius of a few hundred miles is concerned, as to remove, completely, any necessity for demanding, in addition to that, competition between the carriers. The airplane, on the other hand, is making heavy inroads on the long-distance passenger and preferred merchandise traffic, creating a natural competition for that class of business. The coastwise shipping is handling increasing tonnages of a very desirable class of bulk transcontinental traffic and in recent years has cut heavily into the profitable refrigerator business, thus creating a natural competition for that class of business.

All of the aforementioned forms of competition are here to stay. The competition sought in early legislation has been largely, if not completely, supplanted by these other forms of competition, reducing the railroads to the

handling of such mass transportation as can still be performed best by them, and which, no doubt, will continue to be handled by them.

Here again, however, other factors are working against the railroads. The nation has witnessed a formidable movement, decentralizing important industries and establishing manufacturing plants close to the sources of supply, thus greatly and permanently reducing the profitable long-distance railroad shipments of raw materials and finished products, previously enjoyed by the railroads. Not only has this seriously reduced railroad traffic, but a considerable percentage of the traffic that would remain has been taken away by the trucks.

All of these elements must be considered seriously, entirely apart from the present reduction in net railroad revenues due to a reduced volume of business because of the depression, lower rates, and higher wages. The depression is only one of many elements contributing to the present plight of the railroads which has been accentuated by top-heavy financial structures. It can be understood, therefore, that, even with a return to normal times, the railroads will not get even a relative percentage of their former tonnages.

If the railroads were to be placed on a parity with these other forms of transportation in so far as facility ownership is concerned, the public would have to provide either railroad lines or terminals, or both; and it would have to permit private operation over these facilities. Under effective consolidations it would be possible, at least, to have new consolidated terminal facilities owned by the public and leased to the carriers for private operation. This improvement would save the railroads large sums annually and would still permit the continuation of the principle of private ownership and operation.

It has been suggested that transportation companies be organized to supply all forms of transportation. If competition is desired it would be most logical to maintain it under this type of organization, as the present and effective types of competition would be removed. However, there is considerable opposition to this type of organization, particularly by the present competitors of the railroads who would lose their identity.

It must be apparent that the problem which confronts the transportation industry, and which must be solved effectively and courageously, is far greater than the commonly accepted and circumscribed railroad problem. The scope and character of the solution required is so far beyond the control of the railroads that it is impossible for them to attempt a solution alone. Co-ordination and correlation of all forms of transportation, wherein each can labor with a certain degree of freedom within its own sphere, are essential to a sound solution. That requires centralized and authoritative direction and impartial administration.

The proper procedure is to recognize all the foregoing basic elements and prepare a sound, comprehensive plan in the interest of the transportation industry as a whole. The centralized authority could be either: (1) A new Federal department under a Secretary of Transportation; or (2) a Federal Transportation Authority, whose duty it would be to effect a national policy of transportation looking toward a truly co-ordinated and correlated system of transportation for both civil and military purposes, and to promote a plan of sound and all-inclusive railroad consolidation.

The term "military purposes" is included because, although much progress has been made in industrial mobilization for war, nothing has been done to plan and organize properly the transportation system, which is the back-bone of all military operations. The railroads should be prepared to superimpose upon civilian traffic, on a given day, the transportation of vast quantities of raw materials and the finished products of these industrial plants and of the military forces and equipment, without causing a serious disarrangement of civil transportation requirements, or the possible collapse of the entire system. No other major country is as unprepared as the United States in this respect.

Under the aforementioned plan all Federal regulatory functions, covering all fields of transportation, could then be entrusted to the Interstate Commerce Commission, with such added facilities and authority as may be needed.

The writer's views on this general subject, and particularly on the creation of a central Authority, are shared by many outstanding men in different walks of life. W. J. Wilgus, Hon. M. Am. Soc. C. E., has frequently recommended[26] such a central Authority as the only way to cope with this situation adequately. In the report of the Coolidge-Smith Committee (1933), former Governor Alfred E. Smith recommended the creation of a Department of Transportation in Washington, D. C. Similar opinions have been advanced forcefully by Mr. W. M. W. Splawn, Chairman of the Interstate Commerce Commission[27] and (from a legal view) Mr. Leslie Craven, who was for many years a railroad attorney representing various carriers in Washington.[28] This problem was well on the road to a solution when the railroads frustrated their own salvation by having the office of Railroad Co-ordinator abolished.

Mr. Lavis seems to consider centralized planning for the improvement of this industry an un-American procedure—one of governmental paternalism. If the railroads could solve this problem without some centralized direction many would no doubt agree with Mr. Lavis in principle, but since it must be obvious that the carriers cannot do so, it becomes anything but an un-American procedure to help put this industry "on its feet" in the interest of the industry itself and the country at large. Transportation no longer can be viewed as an individual problem, or a private monopoly, because it has long since ceased to be that. This problem must be viewed in its broadest sense, with the realization that one of the fundamental factors, essential to sound and intelligent planning, is unbiased and objective thinking, freed from selfish consideration. It must be solved in the interest of the transportation industry as a whole, not merely the railroads, and the economic well-being of the country in general.

Under the aforementioned circumstances, and restricting comments to the railroad field, with about one-third of the railroad mileage in receivership and with huge sums already advanced to them by the Government, it is difficult to understand how any one can expect to ward off Government ownership of the railroads unless some sound plan of general consolidation is advanced. In fact, some of the foremost engineers, and even railroad and fiduciary officials,

[26] *Military Engineer*, September–October, 1938, p. 349.

[27] Report of the Fiftieth Annual Convention of the National Association of Railroad and Utility Commissioners, New Orleans, La., November 15–18, 1938, *Railway Age*, November 26, 1938, p. 776.

[28] *Atlantic Monthly*, December, 1938, p. 767.

seem to see no other escape than to resort to Government ownership. Competition will probably be restricted or completely removed in certain areas, under Government ownership, unless some effective plan can be devised for private operation of publicly-owned systems, outside of political influence.

*Railroad Consolidation.*—In order to establish a co-ordinated and correlated system of transportation in this country it is essential that each type of transportation, or each system, be given a certain sphere of activity which will enable it to "stand on its own feet." What must be done to accomplish this with the railroads? There is general agreement that one of the fundamental requirements is railroad consolidation.

Apparently, Mr. Lavis is in favor of permitting the carriers to make such voluntary consolidations as they may care to make without regard to some national plan. He believes in maintaining strong competition between the carriers and considers that as creating "a healthy state for an industry such as railroad transportation" (heading "Suggested Remedies: Competition"). Again it must be said that there are many different schools of thought on this subject and it would seem most important to give this matter serious consideration.

As stated previously herein, maintenance expenses will be relatively higher in the future, and it is most unlikely that railroad wages will be reduced, or at least sufficiently to have any appreciable effect upon the net income of the carriers. Therefore, railroads must look elsewhere for important savings.

Such consolidations as have been made in the past have not helped to relieve those carriers that are most in need of assistance, because the strong roads are unwilling to merge with the weak ones. The ultimate result of such a policy will be that the nation will find itself with many poor, broken-down railroads on its hands, which nobody wants, yet which are necessary to serve economic needs. What will then happen to these roads? The result is certain to be more chaos and more demands for Federal help, all at the expense of the taxpayer. It is not likely that the Government, irrespective of who may be in power, will continue to support these roads without demanding a sound reconstitution of the industry as a whole, and the absorption of the weak by the strong, along some kind of national plan.

There are several reasons why little progress has been made in the past by the railroads, or can be made by them in the future, if the problem of consolidation is left in their hands without some form of compulsion following a prescribed pattern.

There are too many divergent interests to be considered, which, if not directed by an impartial outside Authority, will result in disagreement and unsound consolidations. Many high railroad officials would find their positions abolished, and it is only natural for them not to expedite, or even belittle, effective consolidations, although they well know that the industry as a whole would be greatly benefited. Finally, present laws are inadequate to permit really effective consolidations, because they are based upon the predicate that strong competition be maintained, as between the carriers.

With the many forms of natural competition established, as outlined previously herein, there would seem to be no reason for demanding, in addition,

competition between the railroads themselves, except possibly between Mid-Western points and the seaboard. The regions served by the railroads, and the country at large, would be served to much better advantage by removing restrictions as to competition between the carriers and permitting regional consolidations of privately-owned and privately-operated systems. The entire country benefits from a stable transportation industry.

In view of the many competing forms of transportation it must be obvious that the railroad plant, designed and built as the exclusive transport agency, is now greatly in excess of the needs of the people. Some of the facilities can, and should be, abandoned. That can be done under a proper and all-inclusive plan of consolidation. If the railroads do not so consolidate, it will be impossible to abandon a large number of the weak lines which, unfortunately, are essential to the economic existence of many regions.

If the present policy of voluntary consolidations is continued, it must be obvious that a fight for the "survival of the fittest" will result, which neither the railroads, nor the country as a whole, can afford to do. It would result in profits by the few and a "falling by the wayside" of the many, unless, of course, they are saved by the Government; but that is not sound planning, not sound engineering. It would seem far better to plan a sound solution than to sit back and await the inevitable, with resultant chaos.

With increased costs, and increasing competition, railroads of the United States, particularly the weak ones, would have to ask for rate increases at a time when a reduction in rates would have a most wholesome and far-reaching effect upon the entire economic structure of the country.

Where it becomes necessary to abandon certain properties because of the obsolescence of the plant, administrators must also take care of the consequent obsolescence of man-power. If it is economically desirable to abandon certain lines, the carrier can give such labor protection as may be necessary and fair to all. That should be mandatory. It would not be a permanent burden.

In discussing possible losses of labor in connection with such an all-inclusive program it should be stated that it will be necessary to create many new and important consolidated railroad facilities, totaling many hundred millions of dollars, particularly in terminal districts, giving work to scores of thousands of workmen all over the country and greatly benefiting all basic industries. This will largely, if not completely, offset losses of labor due to abandonments which, after all, will represent only a nominal percentage of the total railroad mileage.

It has been recommended recently that the railroads be relieved of certain obligations so that they can spend these savings, amounting to many hundred millions of dollars, for much needed improvements. As some of that money would no doubt be wasted with a proper plan of consolidation, which is certain to be adopted ultimately, it would seem prudent to proceed first with effective consolidations and then make such adequate improvements as will serve such a permanent and co-ordinated railroad plant.

Under the foregoing circumstances it would seem futile to expect any worthwhile solution, unless some central and neutral authority is set up with power to compel proper consolidations. Such Federal Authority must establish the

general pattern, but the railroads themselves should do much of the detail work of effectuating such a plan.

A Possible Solution.—Convinced that the greatest savings could be made under the regional type of consolidation, the writer made a study of consolidating the railroads into six regional systems. That study is presented herein, not as an unalterable plan, but merely to show what could be accomplished under the type of plan recommended in opposition to Mr. Lavis' position with reference to consolidations.

Since many regions have problems peculiarly their own, it is not difficult to group these railroads in such a way as to have each region covered by such a system. Thus the different carriers could be grouped effectively into North Eastern, Central Eastern, South Eastern, North Western, Central Western, and South Western systems, herein referred to as Systems I, II, III, IV, V, and VI, respectively. All traffic within these systems would be handled by that one system, but, as stated before, there should be competition in a larger sense, such as providing, for instance, two competing lines between Mid-Western centers and the seaboard or between important, interior, industrial centers. To establish such competition it will be necessary to provide certain "bridge lines" reaching from one system into such important centers of another system. This principle has been followed herein in the allocation of railroads to the various systems. The ten principal Class 1 railroads tentatively assigned to each of the six systems are as follows:

I.—North Eastern System: New York Central; Illinois Central; Wabash; Pere Marquette; Erie; Boston and Maine; New York, Chicago and St. Louis; Maine Central; Delaware, Lackawanna and Western; and Alton Railroads.

II.—Central Eastern System: Pennsylvania; Baltimore and Ohio; Louisville and Nashville; Chesapeake and Ohio; Reading; New York, New Haven and Hartford; Lehigh Valley; Western Maryland; Virginian; and Long Island Railroads.

III.—South Eastern System: Southern; Norfolk and Western; Atlantic Coast Line; Seaboard Air Line; Central of Georgia; Florida East Coast; Yazoo and Mississippi Valley; Mobile and Ohio; Nashville, Chattanooga and St. Louis; and Gulf, Mobile and Northern Railroads.

IV.—North Western System: Chicago, Milwaukee, St. Paul and Pacific; Chicago and North Western; Great Northern; Northern Pacific; Chicago, St. Paul, Minneapolis and Omaha; Minneapolis and St. Louis; Chicago Great Western; Spokane, Portland and Seattle; Duluth, Missabe and Northern; and, Green Bay and Western Railroads.

V.—Central Western System: Atchison, Topeka and Santa Fe; Union Pacific; Chicago, Burlington and Quincy; Chicago, Rock Island and Pacific; Denver and Rio Grande Western; Western Pacific; Colorado and Southern; Fort Worth and Denver City; Chicago, Rock Island and Gulf; and Northwestern Pacific Railroads.

VI.—South Western System: Southern Pacific; Missouri Pacific; St. Louis and San Francisco; Texas and New Orleans; Missouri-Kansas-Texas; St. Louis Southwestern; Kansas City Southern; Louisiana and Arkansas; Missouri and Arkansas; and Midland Valley Railroads.

No doubt, a further study would reveal the desirability of certain reallocations between the foregoing systems. For instance, System I has no

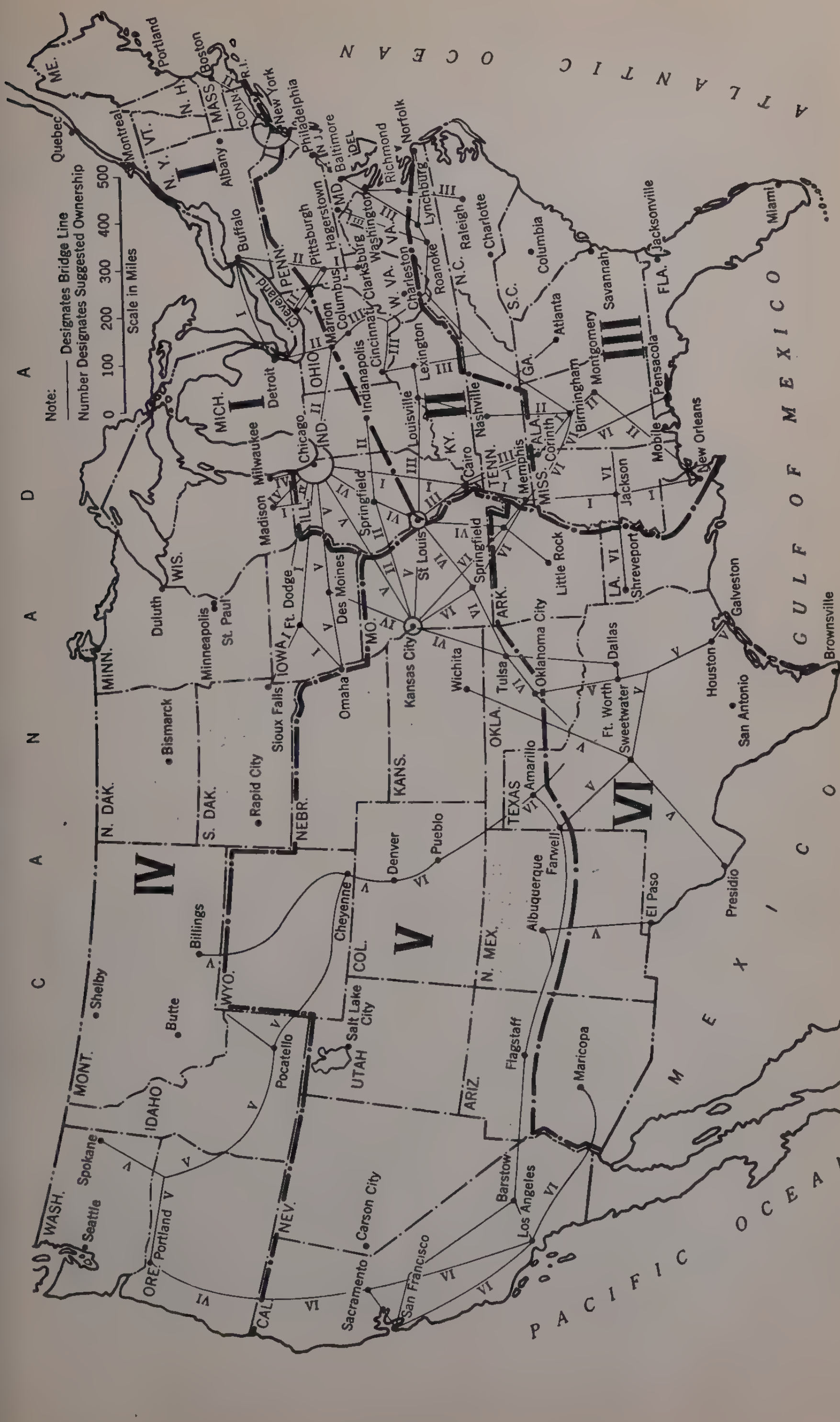
Note: —— Designates Bridge Line
Number Designates Suggested Ownership
0 100 200 300 400 500
Scale in Miles
C A N A D A
A T L A N T I C O C E A N
G U L F O F M E X I C O
M E X I C O
P A C I F I C O C E A N
I
II
III
IV
V
VI
WASH.
ORE.
CAL.
IDAHO
NEV.
MONT.
WYO.
UTAH
ARIZ.
COL.
N. MEX.
N. DAK.
S. DAK.
NEBR.
KANS.
OKLA.
TEXAS
MINN.
IOWA
MO.
ARK.
LA.
WIS.
ILL.
MISS.
MICH.
IND.
KY.
TENN.
ALA.
OHIO
GA.
FLA.
S.C.
N.C.
VA.
W. VA.
PENN.
MD.
DEL.
N. J.
N. Y.
CONN.
R.I.
MASS.
VT.
N. H.
ME.
Seattle
Spokane
Portland
Sacramento
San Francisco
Carson City
Los Angeles
Barstow
Shelby
Butte
Billings
Pocatello
Salt Lake City
Cheyenne
Denver
Pueblo
Flagstaff
Maricopa
Albuquerque
Farwell
Amarillo
El Paso
Presidio
Sweetwater
Ft. Worth
Dallas
San Antonio
Houston
Galveston
Brownsville
Bismarck
Rapid City
Sioux Falls
Omaha
Kansas City
Wichita
Tulsa
Oklahoma City
Duluth
Minneapolis
St. Paul
Ft. Dodge
Des Moines
St Louis
Springfield
Little Rock
Shreveport
Madison
Milwaukee
Chicago
Springfield
Cairo
Memphis
Corinth
Jackson
New Orleans
Mobile
Pensacola
Birmingham
Montgomery
Atlanta
Nashville
Louisville
Lexington
Indianapolis
Cincinnati
Columbus
Marion
Detroit
Cleveland
Pittsburgh
Buffalo
Albany
Montreal
Quebec
Portland
Boston
New York
Philadelphia
Baltimore
Hagerstown
Washington
Clarksburg
Charleston
Roanoke
Lynchburg
Richmond
Norfolk
Raleigh
Charlotte
Columbia
Savannah
Jacksonville
Miami

FIG. 1

access to Baltimore, Md. Either the New York, New Haven and Hartford Railroad may be allocated to System I or that system may be given one of the coal carriers to Baltimore. Such possibilities, however, do not change the principle of regional systems. The regions covered by these systems are shown in Fig. 1.

The total miles of road, the investment in road and equipment, long-term debt, income available for fixed charges, and the fixed charges for each of the proposed six systems, as of the Interstate Commerce Commission Report for 1936, are shown in Table 5. Canadian or Mexican owned roads in the United States are not included.

TABLE 5.—MILEAGES, INVESTMENT, INCOME AND FIXED CHARGES OF THE PROPOSED SIX GREAT SYSTEMS, IN DOLLARS
(Exclusive of Foreign-Owned Roads)

| System | Miles (1) | Investment in road and equipment (2) | Long-term debt (3) | Income available for fixed charges (4) | Fixed charges (5) |
|---|---|---|---|---|---|
| I | 41 997 | 4 109 720 | 2 474 610 | 200 350 | 158 050 |
| II | 36 400 | 4 695 780 | 2 822 880 | 305 170 | 192 650 |
| III | 36 098 | 2 269 870 | 1 064 760 | 96 500 | 62 460 |
| IV | 43 633 | 2 922 740 | 1 658 710 | 80 610 | 78 320 |
| V | 51 467 | 3 294 820 | 1 268 430 | 91 470 | 74 760 |
| VI | 42 587 | 2 617 270 | 1 780 070 | 104 270 | 109 920 |
| Total | 252 182 | 19 910 200 | 11 069 460 | 878 370 | 676 160 |

Table 5 includes all of Classes I, II, III, and switching and terminal railroads. Income available for fixed charges (Column (4), Table 5) was not reported for Classes II, III, and switching and terminal roads; net railway operating income has been used for these cases.

It is practically impossible to have these systems so created as to give each the same mileage and the same income. It may be seen, however, that a fair balance has been achieved in this limited study.

With this plan, and even with many existing top-heavy financial structures and consequent heavy fixed charges, the fixed charges would have been earned in 1936, 1.27 times for System I, 1.58 times for System II, 1.54 times for System III, 1.03 times for System IV, 1.22 times for System V, and 0.95 times for System VI. In every one of these regions there are railroads under reorganization. With these in effect a much more favorable ratio would result.

The study would seem to indicate clearly that the main purpose (namely the establishment of a fair earnings balance) could be obtained which would stabilize the industry and re-establish credit which is so sadly lacking to-day. Such a plan would do effectively what the Recapture Act failed to do.

In addition to this stabilization very substantial operating savings will result from the pooling of equipment and the consolidation of operations within these systems. What these savings will be, of course, is pure speculation based upon hypotheses which, in practice, may have to be materially modified. They have been variously estimated at between 250 and 750 million dollars per annum, based upon depression traffic. Even the lowest value would be a

worth-while inducement for making a change, but the net result is certain to be more nearly 500 million dollars. In addition to this it should be borne in mind that the securities of such a system, with a modified capital structure, would be nearly as safe as Government securities and could be marketed at much lower interest rates than are paid at present. This arrangement would naturally further reduce expenses and still further strengthen the financial position of these systems.

Mr. Lavis seems to be of the opinion that in such large organizations it would not be possible to maintain personal contact. In so far as straight distances are concerned these new systems would be no different from the present large systems reaching from the Mississippi River to either the Eastern or Western seaboard. Instead of one railroad ribbon across this distance there would be several. This will necessitate the creation of regional managers who certainly would be able to maintain a closer contact than is now possible with some of the existing carriers.

Effectuation of Such a Plan.—Within a relatively short period of time, the Federal Authority could prepare a tentative plan and then allow the railroads, say, 6 months or 1 yr, within which to offer modifications and to submit a plan of organization, abandonments, etc. In the meantime, the carriers could appoint some Board of Managers, composed of one member from each road to be consolidated, whose duty it would be to co-operate with the Authority.

After the aforementioned modifications have been considered by the Authority, it could then issue a general plan and order the Board of Managers to manage these properties as consolidated systems for a trial period of, say, 5 yr. This will allow further modifications to be made in the plan and will mark the beginning of 10-yr program of abandonments. These should be spread over a considerable period of years.

After the 5-yr period has elapsed the Authority could then issue its final plan and order the election of system officers and the establishment of these great systems. The Authority would also prepare a financial plan indicating the amount of new system securities it deems proper to allocate to the then existing individual securities. In this allocation weight might be given to: (1) The valuation; (2) the last 5 yr of earnings as an independent carrier; (3) the first 5 yr of system earnings contribution; and (4) other special merits of the respective carriers.

Under such a procedure an immediate beginning could be made and ample time would be allowed for the consideration of varied interests. These six great systems should be organized under a Federal charter as national systems.

*Other Considerations.*—In addition to the consolidation and rate-equalization problems, consideration should also be given to legislation establishing a uniform system of railroad taxation based upon the dictates of national economy and the welfare of the industry, and a uniform system or policy with reference to amounts to be paid by the railroads for grade-crossing eliminations. Numerous other measures, intended to give immediate relief and which have already been recommended by various bodies, should naturally be adopted in addition to the long-term program outlined herein.

*Conclusion.*—The railroads must have new capital to build up these properties. The poor roads, which are most in need of new capital, cannot get it except at exorbitant rates, because the investor, whether it be an individual or a fiduciary institution, is afraid to risk his money.

It is necessary to restore confidence in the industry as a whole. That can best be done by a scientific grouping of poor and prosperous roads within certain regions, with resultant important savings, and the consequent balancing and equalization of resources. It cannot be done by making only a few minor consolidations of desirable properties, as would be the case if left to the railroads for solution. An impartial Federal Authority must be established with power to compel a sound solution in the interest of the transportation industry as a whole and the country in general.

William J. Cunningham,[29] Esq. (by letter).[29a]—A well-rounded and concise summary of the present rail situation is presented in this interesting paper. The writer feels, however, that Mr. Lavis has placeed too much emphasis upon what he calls "fundamental defects in governmental regulation" (heading, "Synopsis"). He is inclined to attribute the difficulties more to regulation than to the loss of traffic because of the general depression. The writer believes that the depression accounts for much the greater part of the difficulties.

The statement that "It has been shown time and time again that the railroads are not over-capitalized" (heading, "Needs of the Railroads: Railroad Capitalization") needs greater amplification than is given in the paper. It is true that they are not over-capitalized from the viewpoint of dollars invested in transportation facilities, but they are over-capitalized from the viewpoint of dollars invested in facilities that are now used and useful. A substantial part of railroad capital is in useless branch lines, needless multiple running tracks, unused sidings, facilities designed for earlier days with shorter train runs and different characteristics of traffic, and duplicate terminals and other facilities. There must be a realistic recognition of the high degree of obsolescence in investment and capitalization written down to conform to the investment in productive property. Holding these views, it is obvious that the writer does not agree with the assertion that "There seems to be no valid argument against maintaining the present financial structures intact" (heading, "The Solution").

Greater emphasis should have been placed upon the need of organized and centralized research. The single paragraph on that subject is not consistent with the statement in the Synopsis that "special stress" is laid on that need.

In the discussion of the causes which have adversely affected the volume of railroad freight business, specific mention should have been made of technological advancements which have made possible the production of more units of finished product for a given volume of raw material. The substantial improvement in the efficiency of fuel consumption on locomotives, for example, has had the result of reducing, correspondingly, the volume of railroad-borne coal for railroad use.

[29] James J. Hill Professor of Transportation, Harvard Univ., Boston, Mass.

[29a] Received by the Secretary December 16, 1938.

S. R. TRUESDELL,[30] Assoc. M. Am. Soc. C. E. (by letter).[30a]—Statistics presented by Mr. Lavis referring to the relative amount of transportation in the United States might have been somewhat more comprehensive as to motor transport if he had used data secured by the U. S. Department of Commerce in its Motor Transport Survey of 1935. This survey showed that common-carrier motor trucks earned $531 000 000, and motor buses $168 000 000, a total of $699 000 000, which with the private truck operations would make a very much more formidable showing than Professor Worley's estimate for 1932. In fact, the Bureau of Economic Research of the Department of Commerce has made estimates from year to year of the earnings of all types of transportation, which show the following distribution for 1936, in millions of dollars:

| Type of transportation | Millions of dollars |
|---|---|
| Steam railroads | 2 787 |
| Motor transport | 1 274 |
| Electric railway | 436 |
| Water transportation | 535 |
| Air and pipe line | 97 |
| Total | 5 129 |

On this basis motor transport earned 25% of the total transportation revenue in the United States for 1936. Similar data in 1929 showed motor transport with $1 194 000 000, or 17.5% of the total transportation bill of $6 847 000 000 in that year. As the actual total in 1936 should have been about $6 000 000 000 on the basis of the index of business activity, the motor trucks have made heavy inroads into railroad traffic during the depression. However, if total traffic had increased in this period in continuation of the trends existing from 1910 to 1929, the increase in truck traffic would have been comparatively unnoticed.

In using the very heavy traffic on the Pennsylvania Railroad between Pittsburgh, Pa., and Altoona, Pa., to illustrate the impossibility of handling rail freight by trucks, Mr. Lavis forgets the tremendous mileage of railroad in the territory west of the Mississippi River which has less than 500 tons per day. It is on these lines that the cost of rail transportation and over-head has risen to 3 cents and 4 cents per ton mile, and thus into the economic realm of the motor truck, which is not 5 cents to 7 cents per ton mile, as cited by Mr. Lavis, but more nearly 3 cents. (Statistics were submitted to the Interstate Commerce Commission by thirteen typical truck operators, in the matter of "Increase of rates in Central Motor Freight Territory, 1937" showing approximately 625 000 000 ton miles costing $19 000 000 or 3 cents per ton mile.) The railroad rate on such traffic is probably only 1.5 cents to 2 cents per ton mile so that truck substitution, which seems inevitable, will be more expensive. The uneconomic and unrecognized result of such transitions is the scrapping of valuable tracks and rights of way for the intangible values represented by

[30] Special Asst., President's Office, C. & N. Ry., Chicago, Ill.

[30a] Received by the Secretary December 27, 1938.

"foot-loose" trucking operators; and this is a result of frozen rates due to over-zealous regulation combined with unreasonable burdens of cost for labor and taxes. Some more modest type of rail motive power could easily accomplish the service needed at a lower cost, provided encouragement could be given to the rail management by relief from taxes, full crew laws, labor agreements, and out-of-date tariffs. Otherwise, the people in the communities served by the rail branch lines will pay increased sums for the transportation of coal, lumber, feed, gasoline, cement, and other necessary bulk products, or they will move away. In most territories affected by rail abandonments both of these contingencies are now happening.

If, through this process of erosion of the feeder rail lines, the railroad main lines are gradually deprived of traffic and, in turn, are compelled to raise rates from the present unprofitable 1 cent per ton mile, where will the process end? Furthermore, are even such apparently essential railroads as the Pennsylvania between Pittsburgh and Altoona safe? In each case in the past, the same conditions were present, the same slow attrition, the same support by regulation to prevent monopoly, and the same results eliminating the stages and the wagon transport of the 1800's, the canals of the 1830's, and now the railroads. Unless this process is to be arrested by release from anti-monopoly regulation, taxes, unreasonable labor provisions, and restrictive rate legislation, no railroad is safe from this uneconomical and artifically fostered process of change.

In regard to Mr. Lavis' paper, some reference is made, or implied, as to the extent of unremunerative capital expenditures since 1920. Mr. Eastman also has referred to this at times as an element for criticism. However, this is only a natural effort of a threatened industry to hold or improve its product in the face of dangerous competition. It would be the same in the case of two competing automobile manufacturers—new plants, new presses, new materials, without one dollar added to profits. If the effort were tardy or insufficient, as in the case of the railroads, there would be an absolute loss or decrease in profits with nothing to show for the capital invested. It is true that some of the expenditures for heavier locomotives, rail, and bridges seem foolish in the light of present facts; but they should be unquestioned as necessary in view of the strife for lower costs to keep the railroad systems solvent. Present-day speeds, quoted by Mr. Lavis as 40 to 50 miles per hr to satisfy the present "hand-to-mouth" merchandising, have probably doubled the costs of the tonnage trains of yesterday at 15 miles per hr; yet eminent economists have repeatedly justified the unit of gross ton miles per train hour (dependent largely on speed) as a measure of railroad efficiency. In the West the longest and most round-about rail lines have the highest "efficiency" in gross ton miles per train hour, and the least profitable results.

This paper has delved deeply and most intelligently into the ills of the railroads. The writer does not want to detract a bit from its value, which is great. He is almost ready to agree that the railroads are going backward; but it is not of their own volition. Railroad morale is very low. Under the stress of straitened circumstances research in both methods and materials is almost nil; and, by and large, the cause for this condition (which is destroying

the largest and at one time the most efficient American industry) is as much from without as from within. It speaks well that there is still continued fighting effort from managements that have been so harassed.

DAVID A. MOLITOR,[31] M. AM. SOC. C. E. (by letter).[31a]—This paper contains a most valuable presentation of the transportation facilities of the United States and deals comprehensively with the problems confronting each class of service. It emphasizes the fact that defects in governmental regulation are responsible for the grave financial difficulties of the railways, and that there should be better co-ordination of all transportation facilities. Finally, it stresses the fact that direct Government aid, like that now being extended to highways, waterways, and airways, should also be provided for the railways.

Informed persons will agree with most of these statements, but when it is proposed to extend Government aid to the railways, much will depend on the particular form in which it is given, whether by an out-right grant, by subsidy, or by an extension of credit; and also, what returns or guarantee should be given to the Government for such aid as may be granted by Congress.

In the case of the highways, the State and National Governments all collect special taxes on gasoline and for owning and operating motor vehicles so that highway construction and maintenance are being paid for by those using the highways. However, the highways belong to all the people, which means the Government, and those contributing the money therefore, have no vested rights in the property.

The natural waterways, in general, were made navigable by improvements and are kept usable by work done under Congressional appropriations paid from any funds in the Treasury. Any individual or corporation owning a boat can enjoy the free use of the national waterways; but again, the users have no vested rights in the property provided by the Government with money from the people. The improvement of the channels through the Great Lakes is the most extensive system in the United States and is not now, nor ever was, undertaken to serve any other purpose than that of navigation for both freight and passengers. In only one or two instances, such as the Tennessee Valley Authority, has flood control and power development been an important factor.

The airways have enjoyed the most generous treatment by the Government since they have been given subsidies toward operating expenses and generally have no monetary investments except in flying equipment.

The case of the railways differs in many respects from that of the other three facilities. It is now (1939) being proposed to place them in a position to partake in some measure of Government aid and thereby correct the inequalities which supposedly exist regarding their ownership, maintenance, and operation when compared with the others.

The railways enjoy no direct Government aid at present, although they carry the mails and considerable Government freight. In most States they pay a property tax similar to private land owners. According to the author, their total property investment in 1920 was about 19 billion dollars to which

[31] Cons. Engr., Harlingen, Texas.

[31a] Received by the Secretary January 6, 1939.

more than 6 billion dollars was added during the next twelve years, making a total of more than 25 billion dollars in 1932. In Table 2, the net operating income for 1932 is given with the annual rate of return on the capital invested, from which it appears that this capital sum was only slightly more than 7 billion dollars. How much of the remaining 18 billion represents property value? How was it acquired and who owns it at present?

Many of the railways received land grants representing enough value to finance their construction with the aid of borrowed money from private investors.

Until about 1920, the railways were in practical control of all transportation business except between such terminals as were also served by the waterways. Their practices were such that they were declared common carriers under control and regulation by the Interstate Commerce Commission as a protection to the public against unfair practices.

The quarrel between the waterways and railways is an old one in which the latter were generally the aggressors. They fought the Panama Canal project for years until there came into the presidency a Theodore Roosevelt, who was big enough to subdue this propaganda and launch the undertaking. The Mississippi River improvement is another case in which a once prosperous water transportation was driven out of business but not with any great benefit to the railways, which were not properly equipped to carry cheap bulk freight.

A case in point to the contrary is the present spectacle of the St. Lawrence-Great Lakes Waterway, for which no one of sufficient stature has yet appeared who could settle the quarrel and subdue the opposition in Congress. It is regrettable that the power development was brought into the "squabble," but the navigation improvement must come sometime, and when the finances of the railways become sufficiently depleted on propaganda, perhaps they will spend what funds may remain on "putting their own house in order."

As long as business was good and money plentiful, due largely to low wages paid to operating employees generally, the railways could afford to fight everybody, including City, State, and National Governments, to maintain supremacy in the transportation field. They could do this and yet pay large dividends to stockholders and high salaries to officials. Then came the war period 1914–20, with the railways enjoying a virtual monopoly, but under-equipped for the huge increase in business, so that with Government control and funds the wheels were kept in motion with sufficient new equipment to handle the heavy traffic, which continued to overtax the railways until about 1926.

The diversion of business to the highways, both passenger and freight, rapidly increased after about 1920, and, with the depression years 1932–38, caused the present surplus of transportation facilities on the part of the railways, while the highways were becoming busier each year.

During recent years the railways have been unable to compete with motor transportation both in kind and cost of service. It is doubtful whether any appreciable amount of the diverted business can ever be recaptured by a highly modernized railway equipment because of the greater convenience and house-to-

house service afforded by motor for both freight and passengers, even at comparable rates.

Since 1928 the writer has traveled 75 000 miles by automobile and had his household goods moved 2 900 miles (including a single move of 1 900 miles) by motor van, all because of the superior efficiency and cheapness of this service. In short, he was prevented from using the railways because of the greater cost involved, their lack of house-to-house service, and loss of time due to poor schedules or infrequent service. The great majority of the traveling public is probably doing the same things for the same reasons.

The existence or non-existence of the railways of the future will hinge on "the survival of the fittest" as has already been the case with respect to interurban railways and some city street-car systems.

Big and little business has sought and found economies in freight costs, as the author mentions, which means a permanent loss to the railways. In their ambition to retrieve some of their lost business and to combat public dissatisfaction, the railways have gone intensively into politics, resulting in much discriminating legislation aimed particularly against highway transportation.

For instance, highway trucks in Texas cannot carry in excess of 7 000 lb and violations are vigorously prosecuted at the public expense to benefit the railways. In other States as much as 20 000 lb is permitted. Recently the Interstate Commerce Commission ruled an increase in billing weights on vegetables effective January 1, 1939, in Texas, New Mexico, Louisiana, and Arkansas, whereas Florida and California, the chief competitors, will not be affected. This ruling will add about $140 per car lot to an already excessive freight rate on shipments from the Rio Grande Valley, which is ruinous to the Valley growers, packers, and shippers alike. It is also proposed to repeal the long-haul and short-haul clause of the Interstate Commerce Commission Act.

Considerable statistical data have been presented by the author, and by others,[32] so that the writer has refrained from making any additions.

The enormous decline in railway business, from whatever cause, has greatly reduced income and thereby depreciated the investment securities. It has destroyed much capital value by mergers, reorganizations, and bankruptcies. What remains of a once lucrative business is now asking for Government aid, preferably in the form of an outright gift, on the plea that the other transportation agencies have enjoyed such aid not hitherto accorded the railways. This is termed the railway problem. Is this the real problem, or is it rather a case of inability to continue to attract investors in a rapidly declining business, which is over-equipped for future needs?

Originally all money invested in railway property, fixed and movable, came from the people. It was not first collected by taxation and then disbursed by the Government, after deducting a large overhead expense. Instead, the business was handled by the railway promoters themselves, and they now wish to sell the property to the Government because it is no longer profitable to operate and hold; but if the Government should buy that which was already created by the people through loss of their invested money and rates paid for

[32] *Proceedings*, Am. Soc. C. E., November, 1938, p. 1731.

service, it would mean that, ultimately, the money has come twice from the people, first to create the property and then to purchase what they have created.

In the case of the highways and waterways the people paid only once, with the Government acting merely as a collecting and disbursing agency.

From the foregoing one must conclude that the railway problem is somewhat akin to that of any large corporate business during a period of obsolescence where the remedy, of course, would be receivership or bankruptcy. Then in the final wind-up, if the Government desired to acquire and operate the property for military or other reasons, Congress could provide the necessary legislation to create a Department of Transportation to handle all such matters, replacing the Interstate Commerce Commission, and, ultimately, to solve the railway labor question which, with other labor legislation, has become a political "foot-ball."

However, if the railways hope to rescue their property they must work out their own salvation by introducing new economies, lowering rates, catering to, and winning, the public by rendering efficient and reasonable service, and finally by recognizing the social implications of their business. The trend toward heavier locomotives and rolling stock, and adding more elaborate passenger trains, is not considered a move in the right direction because of the vastly increased maintenance cost of the road-bed. Lines that cannot exist on the available business income should be abandoned as was done with the interurban lines. Railway employees should realize that they are "killing the goose that laid the golden egg" and should accept reasonable pay reductions.

The foremost competitor of the railway is no longer the waterway but the automotive transportation on the highways. The latter, despite strenuous opposition and propaganda, burdensome traffic restrictions, license fees, and taxes, has become the choice of the people.

Were one to concede that the relations between the various transportation facilities and the City, State, and National Governments do require readjustment to rectify any inequalities supposed to exist, then the following legislative changes by Congress should receive consideration on the assumption that all fixed property be Government-owned and all transportation equipment be privately owned and operated without fees being charged:

(1) The Highways.—Repeal unfair traffic restrictions and remove license fees and gasoline taxes, the Government to build and maintain the highways, which are to be used free.

(2) The Waterways.—National improvements to be provided for navigation purposes and made free to the users.

(3) The Airways.—Remove subsidies.

(4) The Railways.—Acquire road-beds, fixed structures, such as bridges, tunnels, and stations, by purchase or, preferably, by donation. The Government would then build extensions when needed and maintain the property, the operation to be conducted by private capital as at present.

That these changes, or any of them, will ever find favor, or materialize, is highly problematical and much will depend on the funds available to each interest for propaganda purposes. The proposal to vote 6 billion dollars of

credit to the railways would be a dangerous undertaking, to say the least. Although it might relieve the present financial crisis temporarily, it would not solve the railway problem. This sum of money, loaned to the railways, would only add to their present predicament; and it would further reduce the interest on their stocks, since there is no prospect of any increased earning power, and the loan would probably never be repaid.

Years ago the Canadian Government guaranteed certain outstanding obligations of its railways with the result that the properties are now practically owned by that Government. If, and when, any monetary aid is given to the railways, a safe plan would be for the Government to take a first mortgage on the property and thus be in a position to protect its own interests in case of default.

Writers on transportation problems have generally defended or favored only one system as against the others. The writer has attempted to express his unbiased opinion respecting their relations to the public.

C. A. Hoglund,[33] Assoc. M. Am. Soc. C. E. (by letter).[33a]—The suggestion by Mr. Lavis that a study be made of the entire transportation field in the United States is a proper one. This study should be made by men who have: (1) Knowledge of the subject by study or experience; and (2) no personal interest in the result. The entire transportation question should be approached with the thought expressed by Mr. Sheets, as quoted in the paper (see heading "The Money Question: Needed Highway Expenditures"), but with the thought that it should be applied to the entire transportation question instead of to highways. Mr. Lavis' approach to the general subject is somewhat confusing inasmuch as he discusses a special picked problem applying to each method of transportation, to the exclusion of the general problem. He also does not state the problem in such a manner that issues can be drawn as a basis of constructive discussion.

To the writer it seems that the problem should be first considered in a general manner, as for instance: At the present time what are the materials to be moved for the country as a whole? What are the methods and facilities available to move them? What consideration is being given to the relative economy and fitness of the various methods of transportation with a view to encouraging and promoting the use of each for purposes which it can serve best and most economically, while avoiding such use as is merely harmful to agencies better suited to the work?

The definite allocation of materials to be moved by a given type of transportation should be approached with caution, if at all, as the nation has not yet reached a point at which regimentation to this extent is possible. A certain theoretical recommended allocation is probably desirable only as a basis of determining theoretically whether or not there is an over-abundance of classes of transportation in a particular field, and to what extent.

[33] Prin. Engr., Inventory and Pricing Branch, Bureau of Valuation, Interstate Commerce Comm., Washington, D. C.

[33a] Received by the Secretary January 14, 1939.

*Fundamentals.*—Certain fundamentals should be considered before suggesting remedies or solutions for transportation generally. As a background it might be well to review the past and the present quantities to be moved and the classes of transportation.

TABLE 6.—COMMERCIAL FREIGHT TRAFFIC IN THE UNITED STATES
(Revenue Ton-Miles)

| Item No. | Agency of transport | 1926 | 1936 | 1937 |
|---|---|---|---|---|
| 1 | Steam railways* | 447 443 627 000 | 341 181 596 000 | 362 815 382 000 |
| 2 | Inter-city trucks | 23 530 000 000 | 40 230 000 000 | 43 380 000 000 |
| 3 | Great Lakes | 90 037 507 000 | 77 263 645 000 | 93 243 674 000 |
| 4 | Other inland waterways | 9 542 877 000 | 15 387 091 000 | 16 882 943 000 |
| 5 | Pipe lines | 21 700 000 000 | 40 037 134 000 | 44 793 161 000 |
| 6 | Electric railways | 1 312 744 000 | 704 500 000 | 697 482 000 |
| 7 | Airways | Nil | 1 861 000 | 2 156 000 |
| 8 | Total | 593 566 755 000 | 514 805 827 000 | 561 814 798 000 |

* Reports of the Interstate Commerce Commission.

*Volume of Transportation.*—The volume of transportation in 1910 is unknown. The volume in 1926, 1936, and 1937 is given in Table 6.[34] The volume of traffic is less in 1936 and 1937 than in 1926, and in all probability, although the data do not show the volume in 1932, 1933, 1934, and 1935, it will probably be much less than that shown in 1926.

*Changes in Ways of Transportation.*—In 1910 there were approximately the following types of transportation: 240 000 miles of railroad, 200 000 miles of indifferently constructed highways, and about 30 000 miles of pipe lines—a total of approximately 470 000 miles of transportation lines. In 1930 there were 250 000 miles of railroad, 700 000 miles of fairly well co-ordinated highways (perhaps 200 000 of which were concrete or some other hard surface), 110 000 miles of oil and gasoline lines, 30 000 miles of airways, and about 10 000 miles of inland waterways—a total of not less than 1 110 000 miles of transportation lines, some of which were of almost limitless capacity or, rather, such a capacity as it now seems impossible to visualize. The foregoing shows a total increase of 136% in the total length of transportation lines in 1930 over 1910.

*Over-capacity of Transportation System.*—When one refers to the capacity of the transportation system, including ways, methods, and facilities of transport, it will be agreed that as a whole the system is not less than 50% more than that necessary to move the nation's production, using a prosperous year such as 1937 as the measure of an average production year. If this is true, it is axiomatic that the methods of transport should be adjusted and either reduced by law or be allowed to be reduced by competitive action without the restriction of any control.

*Background of the Railroad Problem.*—One of the basic questions affecting the railroads is the following: In 1910 the railroads as a whole were making money and were in practical control of the entire transportation field. There

[34] Report to the President of the United States of a Committee appointed to submit recommendations on the general transportation situation, December 23, 1938, Table 2, p. 66.

were approximately 470 000 miles of transportation lines. The railroads had no competition in the bulk transportation field of which they were not in control through relative efficiency; nor had there been any competition for about thirty years prior to this time. By better and more economical ways and methods the railroads had driven all other modes of transport from the bulk transportation field. To all intents and purposes they were a monopoly. This naturally caused the introduction of Government regulation.

In 1930 a different situation existed, with more than 1 110 000 miles of transportation lines available. The modern highways, airways, waterways, pipe lines, and railroads now give an opportunity for one to choose other ways and methods of transport than the railroads. Highways and trucks of all sizes have opened the transport fields to every farmer and manufacturer in the United States. If a farmer does not like the freight rates, by truck or railroad, on hogs or cattle to the packing houses, he can haul by his own trucks, as illustrated by the fact that, in 1926, 90% of livestock received at 17 large markets were hauled by the railroads and 10% by motor trucks whereas, in 1937, 48% were hauled by railroads and 52% by motor trucks. If the packing houses do not believe that the rates are economical on meats by trucks or railroad, they can transport their own meats. If the steel mills or coal mines do not think the rates to the consumer are economical by the present water facilities, trucks, or railroads, they can transport by their own facilities. The ways of transportation are in existence and available for use, and their capacity has not been reached.

The methods of transport are probably as good to-day (1939) or even better than they were in 1910. Whether or not the methods of one particular class of transportation are more economical than those of another is naturally open to question. The methods and facilities on all of them are now in being and open to development regardless of existence in former times. Therefore, the railroads are in a competitive field, and Government regulation is probably not of as much service as it was in 1910 unless it is extended to cover, not only the ways, but all methods and facilities of transport. It is axiomatic, therefore, that the solution of the problem of one of the types cannot be made fairly without consideration of all classes, methods, and facilities, since they are now more or less competitive.

*Increase in Railroad Capacity of Transporting Materials.*—The speed of freight trains has probably doubled since 1910. As to capacity of locomotives, in 1910 the carriers had a total of 60 019 locomotives with an aggregate tractive effort of 1 588 894 000 lb and an average unit tractive effort of 27 282 lb. In 1935 they had 49 541 locomotives of which 48 477 were steam locomotives, with an aggregate tractive effort of 2 206 201 000 lb and an average tractive effort per unit of 48 367 lb.

In 1910 there were 2 148 478 freight cars with an aggregate capacity of 76 579 000 tons, or an average per unit of 35.9 tons. In 1935 there was a total of 1 867 381 units with an aggregate capacity of 88 677 000 tons, or an average per unit of 48.3 tons.

*Needs of a Defense System in Times of War.*—The problem of a transportation system should never be settled without giving consideration to the fact

that in time of war certain railroads, highways, waterways, pipe lines, and airways will be called upon to bear their part of the transport problem and to this extent these classes of transport are necessary to the national defense system. They should be maintained to a standard necessary to serve their objective. If the business naturally handled and the earnings obtained from this business are not sufficient to maintain them for this objective, they should be maintained with Government assistance.

*Conclusions.*—The foregoing shows that since the time when the railroads were a monopoly in the transportation field there has been an increase of 136% in the length of transportation lines generally. In addition there has been an increase in the efficiency of the railroad transportation to the extent that it would be able to handle perhaps 50% more transportation than it did in 1910, but there is a decrease in the possibilities of obtaining this traffic. The entire railroad field is burdened with over-capacity. What is shown herein relative to railroads is probably equally applicable to highways, waterways, and pipe lines. The crux of the situation—the question of over-capacity of ways, methods, and facilities of transportation—is the real problem.

The writer has purposely omitted reference to the questions of: (1) Unjust railroad taxes; (2) inconsistencies in valuation procedure; and (3) rate of return on carriers' investment.

Item (1).—In 1933 railroads were taxed \$150 000 000 less than in 1926; and in 1938, \$40 000 000 less than in 1926.[35] A further review of their proportion of taxes on their investment will develop the fact that, generally, their tax burden is much less than that now being paid by the ordinary individual.

Item (2).—The value of the railroads as established by the Interstate Commerce Commission, based upon stated elements, is probably correct and proper as applied to a particular railroad. However, there is a condition of over-capacity of transportation facilities, when applied to the railroads in a freight region as a whole. The introduction of this element in the present discussion would probably reduce the values of the railroads in a freight region by not less than 30 per cent.

Item (3).—The propaganda of showing the rate of return on the carriers' investment by including the deficits of poorly paying railroads in with better paying railroads is no more proper than adding the statistics of failures among automobiles, steel, and other corporations to the good ones in obtaining their earnings as an average; and yet, nothing has been mentioned of the fact that the "dividend per cent on railroad stocks paying dividends" has never been less than 4.57% since 1891 (this low value occurred in 1932) and that the average over that period is probably not less than 6 per cent.

[35] Report to the President of the United States of a Committee appointed to submit recommendations on the general transportation situation, December 23, 1938, p. 78.

AMERICAN SOCIETY OF CIVIL ENGINEERS
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DISCUSSIONS

# ANALYSIS OF RUN-OFF CHARACTERISTICS

## Discussion

BY MESSRS. LEROY K. SHERMAN, RICHMOND T. ZOCH, AND MERRILL BERNARD

LEROY K. SHERMAN,[15] M. AM. SOC. C. E. (by letter).[15a]—Since the introduction of the unit graph method,[2] several improvements have resulted, and a keener appreciation of the value of interpreting the hydrograph has developed. The author has presented a useful review of run-off characteristics and has outlined certain original procedures. This discussion will be confined first to brief comments on the fundamental principles and will be followed by a critical analysis of some of the author's conclusions and suggested procedure.

Mathematical derivation of the rising and falling stages of the hydrograph has been studied carefully by several hydrologists. In addition to those referred to by the author, the work of Franklin F. Snyder,[16] Jun. Am. Soc. C. E., published in 1938, should be noted.

Synthetic hydrographs serve a useful purpose. The test of such work is their correlation with observed graphs. When reliable observed hydrographs are available, they should be used in preference to any other basis. Nature integrates all of the factors with no oversights or assumptions.

Transposition of hydrographs may also be useful in the absence of a record. The criticism of the writer's paper on transposition (see heading "Transposition of Hydrographs") is well taken. Smaller areas generally have the steeper slopes. The use of diagrams of distribution percentages on successive days, as illustrated by Fig. 4, should prove useful in many cases.

The time of concentration has been used in many ways. The writer concurs with the definition (see "Synopsis") that concentration, under a continuous rain of uniform intensity, is "the length of time until the run-off becomes constant." The velocity of the stream generally increases with the stage, but

NOTE.—This paper by Otto H. Meyer, Assoc. M. Am. Soc. C. E., was published in November, 1938, *Proceedings*. Discussion on this paper has appeared in *Proceedings*, as follows: February, 1939, by Messrs. Victor H. Cochrane, and Bertram S. Barnes.

[15] Cons. Civ. Engr., Chicago, Ill.

[15a] Received by the Secretary January 10, 1939.

[2] *Engineerings News-Record*, April 7, 1932.

[16] "Synthetic Unit Graphs," by Franklin F. Snyder, *Transactions*, Am. Geophysical Union, 1938, Part I.

it is fairly constant at higher stages. For this reason, best results with the unit graph method are obtained with unit distribution percentages derived from the higher storm intensities. The time of concentration for a given basin, therefore, will be relatively long for the lighter rains. In the case of overland flow, the time of concentration may be materially affected by the original moisture content of soil and vegetation. Variations of from 22 to 50 min have been observed on the same sod covered plots.

Fig. 5 is a constructive suggestion in estimating base flow. The author states (see paragraph before "Conclusions"): "Both methods rely very heavily on correct determination of percentage of run-off and ground-water flow." This truth is worthy of amplification. Net rainfall is not primarily a percentage of rainfall. It is rainfall minus infiltration, after surface run-off begins. Wherever automatic-recording data of rainfall and run-off are available, or wherever the soil infiltration capacity is known, run-off can be estimated accurately by the unit graph or by some other methods (see Table 7). When

TABLE 7.—ALTERNATE METHOD OF ANALYZING A CONCENTRATION CURVE

| Time (ten-minute units) | Rainfall, in inches per hour | Infiltration, in inches per hour | Net rainfall, in inches per hour | Distribution (percentages) | SUCCESSIVE ORDINATES | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | | | | | 10 | 20 | 30 | 40 | 50 | 60 | Total to 60 | 65 | Total to 65 |
| 10 | 1.73 | 0.18 | 1.55 | 35.8 | 0.55 | .... | .... | .... | ..... | ..... | 0.55 | .... | 0.55 |
| 20 | 1.73 | 0.18 | 1.55 | 43.6 | 0.68 | 0.55 | .... | .... | ..... | ..... | 1.23 | .... | 1.23 |
| 30 | 1.73 | 0.15 | 1.58 | 12.0 | 0.19 | 0.68 | 0.57 | .... | ..... | ..... | 1.44 | .... | 1.44 |
| 40 | 1.73 | 0.15 | 1.58 | 5.0 | 0.08 | 0.19 | 0.69 | 0.57 | ..... | ..... | 1.53 | .... | 1.53 |
| 50 | 1.73 | 0.13 | 1.60 | 3.6 | 0.05 | 0.08 | 0.19 | 0.69 | 0.58 | ..... | 1.59 | .... | 1.59 |
| 60 | 1.73 | 0.13 | 1.60 | ..... | .... | 0.05 | 0.08 | 0.19 | 0.70 | 0.58 | 1.60 | 0.29 | 1.60 |
| 65 | Stop | 0.13 | 1.60 | .... | .... | .... | 0.05 | 0.08 | 0.19 | 0.70 | 1.02 | 0.35 | 1.38 |
| .... | .... | .... | .... | .... | .... | .... | .... | 0.05 | 0.08 | 0.19 | 0.32 | 0.10 | 0.58 |
| .... | .... | .... | .... | .... | .... | .... | .... | .... | 0.05 | 0.08 | 0.13 | 0.04 | 0.23 |
| .... | .... | .... | .... | .... | .... | .... | .... | .... | ..... | 0.05 | 0.05 | 0.02 | 0.13 |
| Totals | .... | .... | 10.26 | 100.0 | 1.55 | 1.55 | 1.58 | 1.58 | 1.60 | 1.60 | 9.46 | 0.80 | 10.26 |

only daily quantities of precipitation are given, it is impossible to know whether this rain fell in 2 hr or 10 hr. In this case, the estimate will be more or less approximate and estimated percentages of run-off will suffice.

In Fig. 3, the uniform rate of net rainfall persisted for five units of time and ceased at the time of concentration. The tail of this graph ended six time units after the concentration period, or Time 11. If the rain ceased at the end of Time 2, the figure shows (at the bottom of the hatched area) the termination of run-off at the same point—Time 11. This violates two observed facts in nature: (1) For a uniform rate of rainfall, the time base of the hydrograph for a short rain is less than the time base for a longer rainfall; and (2) for any duration of rainfall the area (volume) of the tail of the hydrograph is equal to the area (volume) of net rainfall, up to said duration, above the hydrograph. This accords with the storage equation.

The author criticizes the unit graph method, particularly with reference to applications extending beyond the time of concentration. He states that the method will follow an actual hydrograph very imperfectly, sometimes giving results in excess of true run-off rates. He proposes another method.

The writer has not observed such discrepancies unless the unit graph method was misapplied or unless the data for a unit or distribution graph were inadequate. The writer presented a hypothetical concentration-time graph in his original paper.[2] He had not, however, tested the method against an actual hydrograph extended beyond the time of concentration. As a check, therefore, he has computed a hydrograph of run-off due to an actual rain of 1.73 in. per hr for 65 min, with a known run-off of 89 per cent. The computations are shown in Table 7. The computed hydrograph is shown in dotted line on Fig. 10. The observed hydrograph, shown solid, is one of a series of tests made

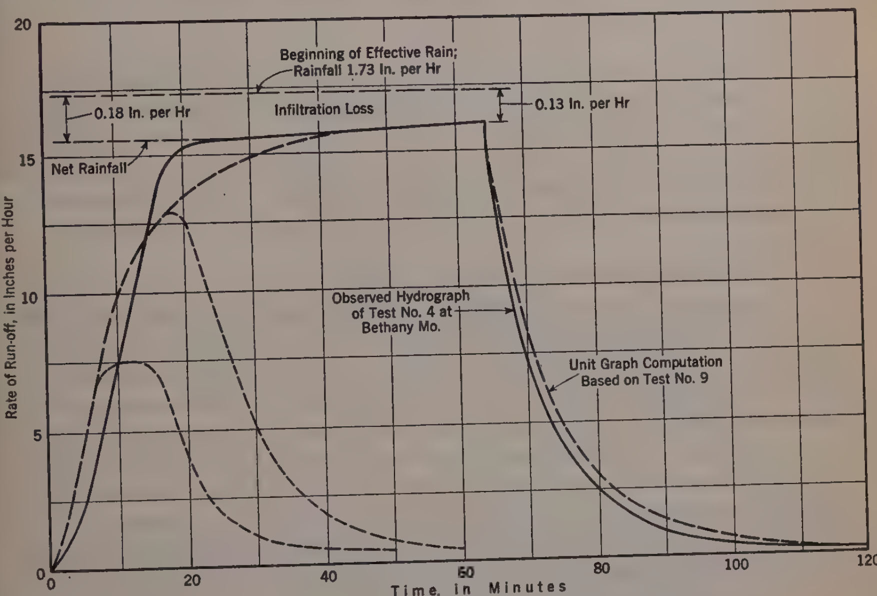


FIG. 10.—COMPARISON OF COMPUTED WITH OBSERVED HYDROGRAPH

in the Fall of 1937 at the Soil Conservation Experiment Station at Bethany, Mo., by Messrs. C. M. Woodruff, Dwight D. Smith and Darnell M. Whitt. A rainfall simulator was used on a plot 7 ft wide and 73.6 ft long. The slope was 7.19% covered with blue-grass sod.

The writer derived a 10-min distribution from an observed hydrograph known as Test No. 4. He applied this distribution to the rainfall data of an entirely different "storm" designated as Test No. 9.

The result speaks for itself and the writer offers no apologies for the unit graph method as a practical working tool.

The observed infiltration rates, as given for Test No. 9, were used. Almost identical results would have accrued had the writer used the average infiltration capacity of 0.158 in. per hr based upon the average of 15 tests made at Bethany.

Apparently Fig. 7 is inconclusive. One peak is checked very well by both methods. The other peak is checked very poorly by both methods. The

writer suggests that if an infiltration capacity had been used, instead of a percentage of run-off, both results would have checked.

In Appendix III, Table 2, the percentage distributions for a hypothetical one-day storm and a three-day storm are given, together with the statement that three days constitute the time of concentration. The writer cannot reconcile these statements. Both conditions cannot be assumed. Each is a function of the one-day distribution. It is to be regretted that the comparison, such as Tables 3 and 4, was not checked against an observed hydrograph.

The author may have developed a desirable principle that would effect an improvement over the unit graph method, but, in the writer's opinion, this has not been demonstrated in the paper.

RICHMOND T. ZOCH,[17] ESQ. (by letter).[17a]—An interesting application of certain theories of rainfall, run-off, and stream flow is contained in this paper, and it is gratifying to find many of the assumptions, upon which Mr. Meyer's paper is based, explicitly stated. In the past, many of the writers dealing with this subject have not stated the assumptions involved nearly as explicitly. It is gratifying also to find the concept of the histogram of the drainage area mentioned.

Mr. Meyer's treatment of the depletion curve (which he calls the storage curve) is theoretically sound, practically convenient and commendable, with the exception that, strictly speaking, the depletion curve begins at a time equal to the concentration time plus the duration of the rain. His treatment of the concentration curve (a section of the hydrograph which he has also named most appropriately) is also theoretically sound and practically convenient; but only so in the section of the hydrograph which comes before the end of the rain, or the concentration time, depending on which of these two times comes first.

Mr. Meyer's presentation is usually clear, but he makes a number of statements which do not seem to be entirely correct. Most of these are in the paragraphs under the heading, "Analysis of the Hydrograph," and it is believed well to point out in detail the statements in which the writer's views differ from Mr. Meyer's. For example, he states, "* * * after a short time lag the run-off will be approximately proportional to net[17b] rainfall," and further, "It is assumed permissible to treat the run-off as a constant percentage of a uniform rain." These statements require much explanation in discussing them. First, it is necessary to distinguish between run-off and rainfall on the one hand, and the rate of run-off and rate of rainfall on the other. The first two are measured in inches and indicate depths of water which have run-off or fallen, respectively, up to time $t$, whereas the other two are measured in inches per hour and are, respectively, the depth of water running off, or falling, per unit time at a given instant of time.

Again, as regards run-off and rate of run-off, it is necessary to distinguish carefully between true surface run-off and underground (internal) drainage. When the soil is not saturated, there is always some infiltration and internal

[17] Meteorologist in Chg. of Library, U. S. Dept. of Agriculture, Weather Bureau, Washington, D. C.

[17a] Received by the Secretary January 13, 1939.

[17b] Correction for *Transactions:* Change "the" to "net," as indicated.

drainage, but the rate of infiltration may exceed the rate of internal drainage. As soon as the soil is completely saturated, there is still infiltration because there is also internal drainage; but the two are equal, and with a uniform continuous rate of rainfall the rate of internal drainage is a constant percentage of the rate of rainfall. Furthermore, after a steady state has been reached (which time may occur simultaneously with, or some time after, but never before, the time of complete saturation of the soil), the rate of the surface run-off is a constant percentage of the rate of rainfall. It is this condition which Mr. Meyer probably had in mind when he made the second statement quoted.

Consider next the capacity of the soil to retain water. The soil may retain water by internal absorption, or it may retain water as surface pondage. In the theoretical treatment of rainfall, run-off, and discharge, it is always possible to distinguish between the internal capacity of the soil and its external (surface) capacity. However, and naturally, the equations involved are more simple if this distinction is not made. Again, from the practical point of view the distinction is certainly not always necessary, because nearly all the internal drainage subsequently appears as discharge. In a stream large enough properly to be spoken of as a river, only the internal drainage in the immediate vicinity (broadly speaking) of the gage will not appear as discharge at the gage. Go to the soil a little distance away from the gage, as well as all the soil still farther up stream, and all the internal drainage, just the same as all the true surface run-off, will appear as discharge at the gage. For this practical reason, as well as the theoretical reason of simplicity, in the following remarks run-off and rate of run-off are considered in the broad sense of the combined surface run-off (or rate of run-off) and internal drainage (or rate of internal drainage).

Consider now the foregoing statements of Mr. Meyer from the point of view of rate of rainfall and rate of run-off. Whether the rate of rainfall is uniform or not, the rate of run-off is never even approximately proportional to the rate of rainfall. After a sufficiently long time (theoretically an infinitely long time—as Mr. Meyer himself states on a later page in another connection—but practically, after the lapse of a number of hours, depending on the nature and condition of the soil), with a uniform continuous rate of rainfall, the rate of run-off must equal the rate of rainfall. This sufficiently long time, of course, must be long enough for the soil to become completely saturated—or waterlogged would probably be a better term—and the rate of infiltration is exactly equal to the rate of internal drainage. At no moment during this period of approach to the time when the rate of run-off will be exactly equal to the rate of rainfall, however, is the rate of run-off approximately proportional to the rate of rainfall. The rate of run-off is proportional, however, to the water remaining with the soil. In many cases, the rate of run-off will be a constant percentage of the water remaining with the soil, although in some cases undoubtedly it is a function of the amount remaining with the soil rather than a constant percentage. The percentage of the rate of rainfall running off at any instant of time steadily increases from the beginning of the rain until a time is reached when the rate of run-off is exactly equal to the rate of rainfall.

The steady state of run-off is that state at which the rate of run-off exactly equals the rate of rainfall. There is also a steady state of discharge, but it always comes some time after the steady state of run-off.

From the foregoing considerations, which are perfectly straightforward and simple, it is possible to set up a differential equation and from it to find the exact mathematical expression which represents the rate of run-off as a function of the rate of rainfall for any given time. This has actually been done by the writer in the paper to which Mr. Meyer refers,[8] and it forms the basis of later papers following it, to which Mr. Meyer does not refer.[18, 19] The differential equation[8] gives rise to an equation which contains the exponential function. This equation probably represents what actually takes place in Nature with remarkable exactness as the time of the steady state is approached; but in the initial range (that is, immediately after the beginning of the rain) it may be only a rough approximation to what occurs in Nature. For this reason the idea of treating the rate of run-off as a function rather than as a constant percentage of the water remaining with the soil has been recognized and, to a certain extent, treated.[19]

Consider next the statements by Mr. Meyer from the point of view of run-off and rainfall. The rainfall that has fallen up to time $t$ is represented by $r\,t$, in which $r$ is the rate of rainfall and $t$ is the time, whereas the run-off which has disappeared up to time $t$ is represented by $(r\,t - r\,c)$, in which $c$ is the capacity of the soil. Clearly, one cannot say that, as the rain continues at a uniform rate (that is, as $t$ increases), the run-off that has disappeared at time $t$ is a constant percentage of the rainfall that has fallen up to time $t$.

The statement, "In actual cases the hydrograph is not expected to vary greatly in shape from one produced by rain of uniform distribution," is one on which the writer is not prepared to comment, but it certainly seems that this statement is worthy of further investigation before being accepted unconditionally.

There is confusion in the paper between the terms "volume of rate of run-off" and "discharge." The latter is the water flowing in a stream and takes account of the time between the moment the water falls to the ground as rain and the instant it reaches the point on the stream at which the discharge is being measured; and this time is a variable quantity increasing with increasing distance from the gage. The volume of rate of run-off is the water disappearing from the soil at any moment of time into the numerous steam channels of a drainage area. Both of these terms are expressed in cubic feet per second, and this may partly account for the confusion between the two. In the statement, "Assuming a uniform, continuous rain, the [discharge] passing any point on the stream will increase as water from more distant parts of the drainage area comes to the point of measurement, * * *," the writer has substituted the word "discharge" for Mr. Meyer's word, "run-off." The discharge does not reach a maximum at, nor does it become constant after, the time of concentration. For a drainage area, the width of which never decreases with

[8] *Monthly Weather Review*, September, 1934.

[18] *Loc. cit.*, Vol. 64, April, 1936; and, Vol. 65, April, 1937.

[19] *Transactions*, Am. Geophysical Union, 18th Annual Meeting, 1937, pp. 425–426.

increasing distance from the gage, this maximum cannot precede the concentration time; but even in drainage areas of this class the discharge does not become constant after this time. For drainage areas, the width of which decreases with increasing distance from the gage, the maximum discharge may precede the time of concentration. The duration of the rainfall itself has an important bearing on the time of the maximum discharge. The duration may be quite short, especially in thunderstorm types of rain, whereas during the winter season the duration may be very long, and the maximum discharge for any uniform continuous rate of rainfall can never precede the time when the rain stops. Hence, if the duration exceeds the concentration time there will be no distinction in the time of occurrence of the maximum discharge arising from different shapes of drainage areas.

Quoting Mr. Meyer: "As the streams then have no source of replenishment other than ground-water flow, which has been eliminated from consideration, all flow thereafter must come from depletion of the water in channel or valley storage." In considering this statement, in place of ground-water flow, which, as Mr. Meyer states, can be eliminated from consideration, eliminate from consideration the stream flow at the time the rain began. By doing this, only the flow due to the rain under discussion is considered. Just the same, much of the stream flow resulting from a given rain is due to water which at some time during the course of its journey from the spot where it fell as rain to the gage on the river was under the surface of the soil. This is true although the soil may have been completely saturated at the beginning of the rain, because even in this case of saturation there is infiltration into the soil, and the rate of infiltration in such a case is equal to the rate of internal drainage. Thus, after a time equal to the sum of the concentration time, plus the duration of the rain, flow results not only from depletion of the water in the channel storage, but also from depletion of the water within the soil which entered the soil during the current rain under consideration.

In the sentence beginning at Line 5 below Equation (3), the word "discharge" occurs four times. This statement is entirely correct if in each case the term "discharge" is replaced by "volume of rate of run-off," or also by "rate of run-off," but, applied to discharge it is never strictly correct from the theoretical point of view, and seldom approximately correct even from the practical point of view.

The statement, "the increase in ordinate of the hydrograph per unit of time is proportional to the additional area, water from which first reached the point of measurement during that unit of time," is only partly correct. Throughout the period of time when the discharge is represented by the curve which Mr. Meyer very fittingly calls the "concentration curve," the discharge is increasing for two different reasons: First, the soil in the drainage area is approaching the limit of its capacity for water and hence a greater and greater portion of the rain falling upon the drainage area is flowing off as free water; and secondly, the area that contributes to the discharge is increasing. After the concentration time, the entire drainage area above the gage is contributing to the discharge, and then the discharge continues to increase, but from the first cause only.

"This break has actually been found on continuous gage records (see Fig. 1), although storms of sufficient duration to produce actual basic hydrographs on large drainage areas are * * *" implies that Mr. Meyer believes that for a rain, the duration of which equals the concentration time, a sharp break occurs in the hydrograph. It is quite true that hydrographs are encountered with such sharp breaks, but these breaks are never due to the concentration time equaling the duration of the rain. They may be caused by moving showers of rain which must move down stream rather than up stream, but a uniform continuous rain will never produce a hydrograph with a sharp break in it, except in the very special case of a heavy rain with a very short duration (practically, the duration of the rain would have to be less than one hour).

Mr. Meyer regards the hydrograph as consisting of only two parts, the first of which he calls by the very descriptive title, the "concentration curve." Generally, for constant rates of rainfall, the hydrograph consists of four distinct parts, which are not in the same order in all cases. Suppose, first, that the duration of the rain is greater than the concentration time. Then the four parts of the hydrograph are as follows: (1) The concentration curve from the beginning of the rain until the concentration time; (2) the saturation curve when the area contributing to the discharge is constant, after the concentration time; (3) the period after the rain stops; and (4) the depletion curve.

Part (1).—During this interval the area contributing to the discharge at the gage is steadily increasing.

Part (2).—After the concentration time, the area contributing to the discharge does not increase, but remains constant; the discharge increases, however, approaching a steady state at which the discharge is equal to the volume of rate of rainfall falling upon the drainage area. During this interval (which, theoretically, may last an infinitely long time, but practically, lasts as long as the rain continues at a constant rate) the hydrograph is represented by the saturation curve, which is asymptotic to the straight line that would represent the discharge at a steady state.

Part (3).—After the rain stops, the discharge is no longer represented by the saturation curve. If a steady state has been reached (practically, for a drainage area large enough properly to be called a river basin, such a steady state of discharge is almost never reached) the discharge decreases immediately. If a steady state has not been reached, the discharge increases after the rain stops until the time of the crest. Here the hydrograph is represented by the transition curve which contains the crest of the flood. The transition curve extends from the end of the rain until a time equal to the duration of the rain, plus the concentration time.

Part (4).—Finally, there is a depletion curve, which extends from a time equal to the duration of the rain, plus the concentration time until, theoretically, an infinite time later, but practically until the beginning of the next rain.

If the duration of the rain is less than the concentration time, obviously, there is no saturation curve. There are still four sections to the hydrograph and a distinction is also caused from the shape of the drainage area. If the drainage area is fan-shaped—that is, if the width of the drainage area increases with

increasing distance from the gage—the maximum discharge must follow the concentration time regardless of how short the duration of the rain may be. In that case there are: (1) A concentration curve from the beginning of the rain until the rain stops; (2) a curve of slow rise from the time the rain stops until the concentration time; (3) a transition curve from the concentration time until a time equal to the duration of the rain, plus the concentration time; and (4) a depletion curve.

If the drainage area is such that its width decreases with increasing distance from the gage, the crest may occur before the concentration time. Then there are: (1) A concentration curve from the beginning of the rain until the rain stops; (2) a transition curve from the time the rain stops until the concentration time; (3) a curve of slow fall from the concentration time until a time equal to the duration of the rain, plus the concentration time; and (4) a depletion curve.

Thus, generally, there are four parts to the hydrograph. If the rain is infinitely short (practically, less than one hour), the hydrograph consists of only two parts: Part (1), a curve of rise, and Part (2), a depletion curve. If the duration of the rain is exactly equal to the concentration time, the hydrograph consists of three parts: Part (1), a concentration curve; Part (2), a transition curve; and Part (3), a depletion curve—the saturation curve being completely absent.

It is for these reasons that the writer believes Mr. Meyer's treatment to be fully the equal of any that has been proposed heretofore for very short rains; but he does not believe it proper to consider Mr. Meyer's "basic hydrograph" as having only two parts. His "basic hydrograph" is naturally divided into three sections and cannot be closely represented by a graph of only two sections.

One point on which Mr. Meyer does not comment (except in the statement which has been quoted about the discharge reaching a maximum and becoming constant at the concentration time) is the determination of the time when the maximum discharge occurs. It has been shown[8] that for a rectangular drainage area, using essentially the same assumptions that Mr. Meyer uses, the time of the maximum discharge is expressed by the function

$$t_c = c \log \left(e^{\frac{L}{cv}} + e^{\frac{t_0}{c}} - 1\right) \qquad (23)$$

In this function $t_c$ is the time of the crest or maximum discharge, measured from the beginning of the rain; $t_0$ is the duration of the rainfall; $L$ is the distance from the gage to the source of the stream; and, $v$ is the velocity of the water. Thus, $\frac{L}{v}$ equals the concentration time, and it should be observed that this function is symmetrical in $\frac{L}{v}$ and $t_0$; and, $c$ is the capacity of the soil to absorb water either internally or as surface pondage. Naturally, with a differently shaped drainage area or with different assumptions of velocity, soil capacity, rates of rainfall and evaporation, this function requires modification. As stated previously, it is possible to separate $c$ into that part which absorbs water internally and the part which is due to surface pondage; but for simplicity they are combined here. Naturally, when they are separated, the function which

represents the time of the crest is a more complicated one, and a simple function is here taken merely to illustrate that, even when these capacities are combined, they influence the time of the crest appreciably, and when they are separated the influence upon the time of the crest is still more marked. This function shows that the time of the crest for this particular shape of a drainage area cannot occur before the concentration time, and neither can it occur before the ending of the rain. The capacity of the soil to retain water is measured in hours, and when the capacity $c$ is said to equal a certain number of hours (say, 10 hr), one means that at the end of 10 hr from the beginning of the rain, the rate of run-off from the soil will be $\left(1 - \frac{1}{e}\right)$ times the rate of rainfall that is falling upon the ground. Thus, the capacity of the soil is a time constant, and is the time at which the rate of run-off reaches 0.632 of its final value. Clearly, this method of expressing the capacity of the soil is simple and convenient. For example, the capacity of the soil may be low. That is, the rate of infiltration may be low, or the soil may be fairly well saturated with water at the beginning of the rain; or, even if no water is on the soil at the beginning of the rain, it may be steep, and hence the surface pondage factor would be low. All of these conditions make the capacity of the soil low, and they also mean that the rate of run-off will approach the rate of rainfall more rapidly than it would if the capacity were high. By measuring the capacity of the soil in time, then, one measures the rapidity with which the rate of run-off approaches the rate of rainfall.

The time of the crest, as represented by Equation (23), is not greatly influenced by the capacity of the soil when the ratio of the duration of the rain to the concentration time is very large or very small (depending, of course, on which is the larger) but, when the duration of the rain and the concentration time are nearly equal, as Mr. Meyer chose them for his "basic hydrograph," the time of the crest is quite sensitive to the varying capacity of the soil. Table 8 (Columns (2) and (3)) illustrates this fact, where the concentration time and the duration of rain are given the same value of 50 hr, but the capacity has the different values shown. The times of the crest given in Table 8, Column (2), have been computed by means of Equation (23).

Naturally, the change in the time of the crest influences the shape of the hydrograph, and this illustrates one of the fundamental and outstanding defects of the concept of the unit hydrograph itself. The unit hydrograph is based upon the assumption that for any drainage area all hydrographs resulting from isolated storms of a given duration have the same shape when reduced proportionally to correspond to the same volume of run-off. Table 8 (Columns (1) and (2)) clearly indicates that the shape of the hydrograph may be decidedly influenced by the different capacities of the soil, although the duration of the rain, the shape of the drainage area, and the depth of the rain are exactly the same.

The mathematical theory shows that the concept of the unit hydrograph is practically applicable when: (1) The concentration time is appreciably greater than the duration of the rain and the drainage area is of such shape that the crest cannot precede the concentration times; and (2) the duration of the rain is

appreciably greater than the concentration time, regardless of the shape of the drainage area. Under other conditions, the application of the unit hydrograph will not yield reliable results.

TABLE 8.—Influence of Soil Capacity on Time of Crest

| Capacity of soil, in hours | Rectangular Drainage Area | | | | Triangular Drainage Area | | | |
|---|---|---|---|---|---|---|---|---|
| | Duration, Fifty Hours | | Duration, Five Hours | | Duration, Fifty Hours | | Duration, Five Hours | |
| | Time of crest, in hours | Maximum discharge in mile-inches per hour | Time of crest, in hours | Maximum discharge in mile-inches per hour | Time of crest, in hours | Maximum discharge in mile-inches per hour | Time of crest, in hours | Maximum discharge in mile-inches per hour |
| (1) | (2) | (3) | (4) | (5) | (6) | (7) | (8) | (9) |
| 10 | 55.89 | 862.2 | 50.04 | 99.2 | 51.79 | 929.7 | 20.52 | 127.9 |
| 20 | 63.02 | 739.6 | 50.46 | 90.8 | 55.26 | 800.7 | 27.61 | 99.6 |
| 40 | 71.54 | 569.2 | 51.50 | 70.0 | 61.16 | 603.4 | 34.95 | 70.2 |
| 50 | 74.49 | 510.2 | 51.90 | 62.0 | 63.30 | 538.8 | 37.19 | 61.2 |

The influence of the time of the crest and the capacity of the soil upon the magnitude of the maximum discharge is also great. Column (3), Table 8, illustrates the corresponding values of the maximum discharge from a rectangular drainage area of 10 000 sq miles and 200 miles in length, and for a rate of rainfall of 0.10 in. per hr. These maximum discharges are computed from the formula

$$y_c = W\, r[L - v\,(t_c - t_0)] \ldots\ldots\ldots\ldots\ldots\ldots (24)$$

in which $r$ is the rate of rainfall, and $W$ is the width of the drainage area. In the foregoing cases, had the rain continued sufficiently long to bring the entire drainage area to a steady state (which, necessarily, would be longer than the time necessary to bring an individual unit to a steady state) the discharge clearly would be 1 000 mile-in. per hr.

The discharges for the corresponding capacities are expressed in mile-inches per hour, and 1 mile-in. per hr can be converted to cubic feet per second, thus:

$$1 \text{ mile-in. per hr} = \frac{5\,280 \times 5\,280}{12 \times 60 \times 60} \text{ or } 645.3 \text{ cu ft per sec.}$$

For short rains, the time of the crest does not vary appreciably with a varying capacity of the soil, provided the shape of the drainage area does not permit the time of the crest to precede the concentration time, but there still will be an appreciable variation in the maximum discharge. Thus for the same conditions that were outlined for Columns (2) and (3), but for a rain of only five hours' duration instead of fifty hours' duration, the corresponding values are shown in Columns (4) and (5), Table 8.

It should be emphasized that for Columns (2) to (5), Table 8, the rate of rainfall is the same, namely, 0.10 in. per hr. For Columns (2) and (3) the duration is 50 hr, so that the rainfall is 5.00 in.; and, for Columns (4) and (5), the duration is only 5 hr, so that the rainfall is only 0.50 in. Evaporation after the rain stops has been neglected, but formulas to correct the maximum discharge for this evaporation are readily obtainable.

By changing the shape of the drainage area, different values again result. Equations (23) and (24) are applicable only to a rectangular drainage area.[20] For a drainage area with a width $W(x)$, in which $x$ is the distance above the gage and $W(x)$ a function that can approximate the histogram of the drainage area as closely as desired, the maximum discharge is given by

$$y_c = r \int_{v(t_c - t_o)}^{L} W(x)\, dx \qquad (25)$$

for a duration of rain exceeding the concentration time regardless of the shape of the drainage area, and also for short rains if the drainage area is fan-shaped; or by a similar integral with upper limit $v\, t_c$, if the duration of the rain is less than the concentration time and the width of the drainage area decreases with increasing distance above the gage. Naturally, Equation (25) requires modification to apply it to varying conditions of soil capacity, rates of rainfall, and velocity of water.

To illustrate further how different soil capacities give rise to hydrographs of different shapes for the same drainage area, even if the rain is uniform throughout its duration, consider a hypothetical triangular drainage area, also of 10 000 sq miles.[20] For a long rain, duration of 50 hr, the appropriate formulas indicate the values given in Columns (6) and (7), Table 8. For a short rain, duration of 5 hr, the time of the crest precedes the concentration time, with the results shown in Columns (8) and (9).

Merrill Bernard,[21] M. Am. Soc. C. E. (by letter).[21a]—The unit hydrograph, as presented by Mr. Sherman[2] in 1932, has sustained itself well as the basis of a rational theory of relationship between rainfall and stream flow and as a practical tool for the solution of important hydrologic problems that previously had depended largely upon restricted, inflexible, empirical methods. Contributions following Mr. Sherman's introduction of the idea, including that of the author, have injected refinements into the technique and have added to the general knowledge of basic hydrologic relationships.

The stimulus given the field of applied hydrology by the unit hydrograph concept has created general dissatisfaction among hydrologists with the basic precipitation data, emphasizing, as it has, the inapplicability of average 24-hr rainfall to problems on the smaller water-sheds. The result can be expected, in the near future, to take the form of co-operative effort toward a network of recording rain gages over the United States, the beginning of which is evidenced in the 140 such stations established in 1937 by the Commonwealth of Pennsylvania.

The paper by Mr. Meyer has merit in a freshness of viewpoint, in the successful attempt to express the hydrograph of stream flow mathematically, and in the demonstration of the need for refinement in basic hydrologic data.

[20] "Run-Off—Rational Run-Off Formulas," by R. L. Gregory and C. E. Arnold, Assoc. Members, Am. Soc. C. E., *Transactions*, Am. Soc. C. E., Vol. 96 (1932), Fig. 5, p. 1068.

[21] Chf., River and Flood Div., U. S. Weather Bureau, Washington, D. C.

[21a] Received by the Secretary January 19, 1939.

[2] *Engineering News-Record*, April 7, 1932.

Refinement in method, as the writer conceives it, must evidence itself in two ways: (1) By producing consistently better agreement between computed and observed values; and (2) by adhering more rationally to the phases of the natural phenomenon of stream flow produced by surface run-off from rainfall.

The mathematical synthesis of the hydrograph has a definite place in this field, for the hydrograph is, in fact, a series of related curves capable of being expressed by three or more mathematical equations, the factors in which can be taken to represent the causes that combine to produce the hydrograph. A fuller understanding of the theory will depend upon the ability of hydrologists to inject greater detail into analysis, the confirmation of which must await the availability of correspondingly refined data.

Considering a practical relation between the adopted unit of time and the area of a water-shed, the degree of refinement will vary directly with the reduction of the time unit and will be proportional to the extent that the variation in rainfall over the water-shed and throughout time can be taken into account. Another source of appreciable error which, it is hoped, may be removed soon, lies in the inability of hydrologists to separate, accurately, the stream flow produced by surface run-off from that originating in ground-water sources.

It is difficult to associate the author's aims with his generalized assumptions. One can agree that total storm rainfall and total storm run-off may be expressed as a ratio. However, as far as the writer knows, the only approximate percentage through time is that existing between rainfall and the hydrograph of flow after the rainfall has been distributed as it will contribute to the flow at the outlet (the pluviagraph), and not in the order of its fall. It is more likely that refinement will take the course of dealing with the losses to run-off as a deduction from rainfall than as a ratio of run-off and rainfall.

Before continuing an examination into the author's work, it seems desirable to consider the least understood and most loosely expressed factor entering the synthesis of the hydrograph from rainfall—"concentration time." In its first conception this was considered to be the time in transit (based on the average velocity of flow) from the most remote point of the water-shed to the outlet. Later it was conceded that distance from the remote point to the outlet must be measured in units of time rather than in units of length; so that now "concentration time" is more generally understood to be the time interval at the end of which all parts of the water-shed are contributing to peak flow. In unit-hydrograph studies the concentration period is shown as the time interval between the respective centers of mass rainfall and resulting hydrograph, which is closely related to the peak ordinate of the hydrograph.

The author describes the time of concentration as that at the end of which the run-off becomes constant. Does he mean run-off (inflow) into storage, or discharge (outflow) out of storage? If he means the former, it would seem that wide fluctuations in the rate of rainfall must preclude the possibility of inflow prevailing at a constant rate for any appreciable time. If he means the latter, the assertion is not borne out in the shape of the normal hydrograph.

First, it must be understood that the concentration period cannot be a constant factor on any water-shed; it must vary according to the limits fixed

by surface cover, the channel characteristics of the collecting system, and the distribution of rainfall over the area. Time of travel through the system of channels varies inversely with depth and effective hydraulic radii, fair stability being reached in both average velocity and wave velocity as bankful stages are approached.

If the time of concentration is considered to be that at the end of which all parts of the water-shed are contributing to peak flow, one must still be critical, because it is known that on a particular basin rainfall may be so favorably concentrated over the lower principal tributaries as to create the peak of the hydrograph before outlying areas have produced and delivered effective quantities of run-off to the outlet. Particularly on larger basins it is probable that surface retention and retardation will prevent contributory flow from appreciable areas during the peak stage, withholding their contribution to sustain the hydrograph in its recessional phase.

The writer has felt for some time that there is definite place for observational research in any plans there may be, in order to refine one's knowledge of rainfall-run-off relationships; and it seems to him that, with a little exercise of the imagination, the phenomenon of flow assembly can be explained as follows:

Select a unit of area on a natural water-shed of such size that a detailed view of the whole may be had from a central position, say, an acre. Take a position at the beginning of a downpour and note that the water at first soaks into the ground. Water begins to accumulate in the depressions and soon overflows and combines into a system of heretofore indiscernible small channels. The run-off from the unit area accumulates rapidly at the lowest corner where it enters a small, but well-defined, channel. The map of the water-shed indicates plainly the course the water will take to reach the outlet.

On another occasion one will observe an application of rainfall throughout a unit of time to the unit of area. Where, in the previous experience, the ground at the beginning of the rain was quite dry, with a resulting prolongation of the period necessary to satisfy the infiltration capacity of the soil, in this case antecedent rainfall has largely satisfied this initial condition, so that the run-off begins quickly, and assembles and discharges at a higher rate than before.

The observer visits his unit of area on numerous occasions and notes that, although there are obvious differences in the amount, and in the rapidity with which run-off flows to the outlet and joins the flow in the small outfall channel, there is a marked stability in the regimen of flow assembly. Run-off begins, combines, discharges from the area, and reaches the outlet in about the same manner from each rainfall of unit-time duration.

It is not difficult, then, to envision a water-shed comprised of unit areas which are related in the common characteristic of the time taken by their run-off waters to reach the outlet. In a natural water-shed these units would arrange themselves with reasonable continuity along lines conforming fairly well to surface contours. The isochrons, or "time contours," would differ from the surface contours in that, whereas the latter would be roughly parallel and would reach up into the principal tributaries of the system, the isochrons would be

foreshortened because of the much higher rate of travel of the water after it has reached the principal channels.

To organize a mind picture of the process along these lines consider:

(1) A water-shed reflecting normal regional characteristics;

(2) A surface contour entering the water-shed at a line normal to the outlet;

(3) An isochron connecting the outlets of contiguous unit-areas entering the water-shed at the same point as the surface contour but bearing down-slope as it continues into the water-shed in order to maintain a constant delivery period (thus it provides for the obvious necessity of reducing the distance from the unit-area to the channel system in a direction normal to the latter in order to compensate for time of travel through the channels to the outlet); and,

(4) A map of water-shed, developed through acceptable analytical procedure (and based, if possible, on observational data) showing the unit-areas having a common time of travel from their outlets to that of the water-shed arranged along isochrons at fixed-time intervals, as the hour.

Concentration time may be expressed as the time required by the first water leaving a series of unit-areas along an isochron to reach the outlet; or it may be considered as the time taken by a series of unit-areas to discharge, completely, the run-off resulting from a unit-time rainfall. It is better understood as an enveloping line of equal time under which all of the units are contributing to the flow at the outlet within the period designated by the isochron. If the map shows lines of equal time of concentration at hourly intervals, and the areas progressively enveloped by each are determined, one can say that at the end of the first hour so many acres will be contributing to the flow at the outlet; at the end of the second hour so many more units, or a total of so many units will be contributing—and so on to the outer limits of the water-shed. Thus, in the isochronal map, one discerns the principal reason for the constancy in the time-base of a hydrograph resulting from rainfall confined to a unit of time.

The unit hydrograph has at least three phases—the rising, the peak, and the recessional. Thus far this discussion has had to do with the rising and peak stages. The rise is accounted for by the fact that the flow at the outlet is increasing because of (*a*) an increasing depth over the unit-areas already contributing to the outlet flow, and (*b*) an increasing contributing area. The latter influence on accelerated flow is removed when elapsed time has progressed upward and over the isochronal map to the outer limits of the water-shed. If the cessation of rain is coincident with the end of this time-period the result is a unit-hydrograph as conceived by Mr. Meyer. There are two conditions, however, that must be satisfied before the analyst can be assured that the hydrograph will peak at, or shortly after, this time interval, and not before. First, the water-shed and the collection system must have fair uniformity in shape and in pattern. That is, the isochrons must be fairly regularly spaced and not concentrated around several important contributory branches discharging near the outlet and thus leaving appreciable areas at the outer limits of the water-shed to be represented by widely spaced and comparatively few isochrons. Second, the rainfall must be uniform throughout the time interval

and over the water-shed; because, if normal variations are considered, it is obvious that heavy concentrations in the early part of the period and over the lower part of the water-shed will create the peak of the hydrograph before the more remote parts are contributing. Continuing rainfall will build up a gradual increase through the peak phase until a period of rainfall equal to the time-base of the unit-hydrograph has passed, after which the peak will increase or decrease with the fluctuations in rainfall intensities to the end of the rainfall period. The hydrograph now starts downward through the recessional phase which constitutes withdrawal from the water in storage in the channel system.

Concentration time has been discussed as if it were a constant characteristic of a water-shed. This is true only within limits that are believed to be relatively unimportant when other errors inherent in hydrologic data themselves are taken into account. Variations in the concentration period are attributable to the following causes:

(1) Seasonal and developmental changes in vegetal cover of the water-shed affecting the quantity and rapidity of run-off;

(2) Seasonal changes in bank and channel growth affecting the movement of water in the channel system;

(3) The initial depth of water in the channels at the beginning of run-off; and,

(4) Variations in the distribution of rainfall over the water-shed as these variations affect the assembly of flow and propagation of the flood wave which delineates the hydrograph at the outlet of the water-shed.

If the actual condition of flow during a period of flood run-off is considered, one can appreciate the inappropriateness of applying average velocities to the moving water. The synchronization of flow assembly on a drainage area of fixed regimen varies only slightly from flood period to flood period—a fact which finds support in the similarity in shape of unit-hydrographs of a particular water-shed.

Likewise, the conditions of the storage equation must be met (that is, within a unit of time, inflow plus or minus changes in storage must equal outflow). Inflow is a highly variable factor, being the sum of innumerable and widely varying contributions to flow discharging into channel storage along both banks of the entire channel system. Outflow is much less erratic in fluctuation, rising to and receding from a crest stage with the regularity of a compounded mathematical curve. Therefore, the balance demanded by the storage equation must be maintained by rapid changes in the configuration of the storage volume, creating irregularities in hydraulic slope and channel section which are recognized as characteristic of transitional waves.

Correction for *Transactions:* Page 1780, Line 17, after "flood wave" insert Footnote 8*a*—"*Transactions*, Am. Soc. C. E. (1936), p. 189."

AMERICAN SOCIETY OF CIVIL ENGINEERS
Founded November 5, 1852
DISCUSSIONS

# TESTS ON BUILT-UP COLUMNS OF STRUCTURAL ALUMINUM ALLOYS

## Discussion

By Alfred S. Niles, Assoc. M. Am. Soc. C. E.

Alfred S. Niles,[12] Assoc. M. Am. Soc. C. E. (by letter).[12a]—From two distinct viewpoints, Mr. Holt's paper is a valuable contribution to the literature of column design. Its most obvious merit is that it extends the knowledge of column behavior to large members of aluminum alloy. It also throws additional light on some problems of column design that are of interest to the engineer regardless of whether he intends to use aluminum alloy or steel as his structural material. The discussion of the stresses in lacing-bars, for example, should be of as great value to the designer of steel columns as to the worker with aluminum. In the past this problem has been slighted and lacing-bars have been designed by methods based on so many assumptions and on so few experimental data that they scarcely rise to the dignity of being called empirical, but can only be described as rule of thumb. The more information that can be obtained on this and several of the other topics treated in the paper, the more efficiently will the engineer be able to design his columns, and the more safely can he use unconventional sections for special purposes.

One phase of the tests that is rather meagerly covered in the paper is the tendency of the "cross" and the "tee" sections to fail by twisting. In Table 3 four columns are listed for which failure by twisting was predicted, and in each case the prediction was fulfilled. It may be noticed, however, that there is no mention of predicted loads at which failure in this manner would take place. Furthermore, it should be noted that three of these columns failed under much lower average stresses than those predicted by Equations (3) and (5). It might be thought that Equation (6) would be helpful in predicting the probable stress at failure by twisting, since buckling of an outstanding leg due to local instability would tend to cause the columns in question to twist. That this

Note.—This paper by Marshall Holt, Jun. Am. Soc. C. E., was published in November, 1938, *Proceedings*. This discussion is printed in *Proceedings* in order to bring the views expressed before all members for further discussion of the paper.

[12] Prof., Aeronautic Eng., Leland Stanford Junior Univ., Aero. Laboratory, Stanford Univ., Stanford University, Calif.

[12a] Received by the Secretary January 13, 1939.

equation does not apply is evident from the fact that two of these columns failed by twisting under much higher unit stresses than the larger values obtained from that formula. The real situation is that, for any column there is a critical load which cannot be exceeded without producing general instability with respect to torsional failure. For most of the standard column sections this load is so much larger than the critical loads corresponding to other types of failure that it need not be considered in design. For the "cross" and "tee" sections, however, it is likely to be less than the other critical loads and becomes a limiting factor. This type of failure is not discussed in the paper although the aforementioned predictions show that the author was properly aware of its likelihood. Its importance to the designer is indicated by the relatively low average stresses at which the first three columns listed in Table 3 failed.

One probable reason for the lack of predictions of the stresses at which torsional failure was to be expected is the present very primitive state of the theory applicable to this phenomenon. Although it had been noticed in a number of earlier column tests it was not until quite recently that any notable progress was made in explaining it rationally or in predicting the loads at which it might be expected to occur. At present the best discussion of the phenomenon easily available to American engineers is a report by E. E. Lundquist, Jun. Am. Soc. C. E., and Mr. C. M. Fligg[13] of the engineering staff of the National Advisory Committee for Aeronautics. The theory of this report, however, is incomplete since it is limited to the case of a column with a doubly symmetrical cross-section, and is not supported by test data.

Corrections for *Transactions:* In Table 1, Columns (9), (10), and (11), Specimens Nos. 2C-3 and 2C1P-2, delete all decimal points; in Fig. 4, curve for Bar 20, change "Equation (5)" to "Equation (3)"; in Line 7 below Equation (4), change "Equation (1*a*)" to "Equation (1*b*)"; in Table 3, in heading to Column (2), delete asterisk and in heading to Column (3) change "Equation (3)" to "Equation (4)"; in Fig. 7 change abscissa caption "Depth of Buckle, in Miles" to "Depth of Buckle, in Inches"; in Line 1 below Figs. 6 and 7 change "Fig. 2(*b*)" to "Fig. 2(*a*)"; in Line 21, page 1806, change "Fig. 2(*b*)" to "Fig. 2(*j*)"; and in the Appendix, definition for *a*, change "Equation (3)" to "Equation (6)."

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[13] "A Theory for Primary Failure of Straight Centrally Loaded Columns," National Advisory Committee for Aeronautics, *Technical Report No. 582*, Washington, D. C., March, 1937.

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DISCUSSIONS

# DESIGN OF DOWELS IN TRANSVERSE JOINTS OF CONCRETE PAVEMENTS

## Discussion

BY L. E. GRINTER, M. AM. SOC. C. E.

L. E. GRINTER,[11] M. AM. SOC. C. E. (by letter).[11a]—The paper by Mr. Friberg presents the second restatement of the general principles of dowel design first presented briefly by the writer as one phase of the structural design of concrete pavements.[12] In 1932 R. D. Bradbury,[3] Assoc. M. Am. Soc. C. E., made use of these same principles to draw other conclusions as to the proper size and spacing of dowels. Since the writer cannot avoid the responsibility of having suggested these approximate methods of analysis to the profession, it seems necessary to point out some of the limitations that should be placed upon the use of such approximations before persons without a proper understanding of the problem reach unacceptable conclusions.

*Change in Dowel Practice.*—Mr. Friberg states that present practice is to place $\frac{3}{4}$-in. or $\frac{7}{8}$-in. dowels of 2-ft length at from 12-in. to 20-in. centers. When the writer made his study of this problem,[12] a common practice was to use $\frac{1}{2}$-in. or $\frac{5}{8}$-in. dowels at from 3 ft to 5 ft centers. Since that study showed the certain need for heavier dowels at much closer spacing, the trend in present practice is decidedly gratifying; but the trend toward the use of heavier dowels more closely spaced must be carried further and such dowels may need to be made longer than 2 ft for maximum efficiency, as will be demonstrated. This appears in conflict with Mr. Friberg's conclusion that dowels may be shortened to less than 2 ft. Naturally, this lack of agreement is produced by different points of view regarding $K$, the foundation modulus.

*Dowel Deflection Curve.*—Based upon mathematically derived formulas that assume perfect elasticity, originally published by Professor S. Timoshenko and

NOTE.—This paper by Bengt F. Friberg, Assoc. M. Am. Soc. C. E., was published in November, 1938, *Proceedings*. This discussion is printed in *Proceedings* in order that the views expressed may be brought before all members for further discussion of the paper.

[11] Dean, Graduate Div., and Director of Civ. Eng., Armour Inst. of Technology, Chicago, Ill.

[11a] Received by the Secretary December 8, 1938.

[12] "Design of the Reinforced Concrete Road Slab," by L. E. Grinter, M. Am. Soc. C. E., *Bulletin No. 39*, Texas Eng. Experiment Station, pp. 53–59, 75, and 79–83.

[3] "Design of Joints in Concrete Pavements," by R. D. Bradbury, Assoc. M. Am. Soc. C. E., *Proceedings*, Twelfth Annual Meeting, Highway Research Board, 1932.

TABLE 8.—COMPUTATIONS OF RELATIVE DOWEL DEFLECTIONS

| ANGLE $\beta x$ | | $\beta x$ (times $\pi$) | $e$ | $\cos \beta x$ | $\sin \beta x$ | $\cos \beta x - \sin \beta x$ | DEFLECTIONS | |
|---|---|---|---|---|---|---|---|---|
| Degrees | Radians | | | | | | $y_P$ | $y_{M_0}$ |
| 0 | 0.000 | 0.00 | 1.00 | 1.0 | 0 | +1.0 | +1.0 | +1.0 |
| 30 | 0.166 | 0.52 | 0.59 | 0.866 | 0.5 | +0.366 | +0.51 | +0.216 |
| 45 | .... | .... | .... | 0.707 | 0.707 | 0.0 | .... | 0.0 |
| 60 | 0.333 | 1.05 | 0.35 | 0.5 | 0.866 | −0.366 | +0.175 | −0.128 |
| 90 | 0.500 | 1.57 | 0.21 | 0.0 | 1.0 | −1.0 | 0.0 | −0.208 |
| 120 | 0.666 | 2.10 | 0.122 | −0.5 | 0.866 | −1.366 | −0.061 | −0.167 |
| 150 | 0.832 | 2.62 | 0.074 | −0.866 | 0.5 | −1.366 | −0.063 | −0.100 |
| 180 | 1.000 | 3.14 | 0.043 | −1.0 | 0.0 | −1.0 | −0.043 | −0.043 |
| 210 | 1.166 | 3.67 | 0.026 | −0.866 | −0.5 | −0.366 | −0.022 | −0.009 |
| 240 | 1.333 | 4.20 | 0.015 | −0.5 | −0.866 | +0.366 | −0.007 | +0.006 |
| 270 | 1.50 | 4.72 | 0.009 | 0.0 | −1.0 | +1.0 | 0.0 | +0.009 |

Mr. J. M. Lessels,[13] the writer has made the computations of Table 8. In addition to Equation (1), the formulas needed are:

$$y_P = \frac{P\, e^{-\beta x}}{2\, \beta^3\, E\, I} (\cos \beta\, x) \quad \ldots\ldots\ldots\ldots\ldots\ldots (19a)$$

and,

$$y_M = \frac{M_0\, e^{-\beta x}}{2\, \beta^2\, E\, I} (\cos \beta\, x - \sin \beta\, x) \quad \ldots\ldots\ldots\ldots\ldots (19b)$$

in which $y_P$ is the deflection at any distance, $x$, along the dowel due to the load, $P$, applied at its end, and $y_M$ is the corresponding deflection due to an applied end moment, $M_0$. The constants, $E$ and $I$, belong to the dowel since the sub-

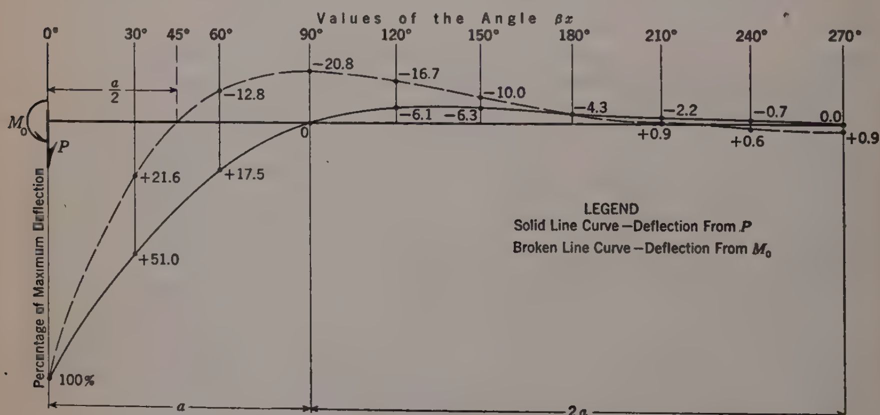


FIG. 8.—COMPARISON OF DOWEL DEFLECTIONS FROM LOAD AND MOMENT

grade factor, $K$, includes, in its complex make-up, the modulus of deflection of the sub-grade, the modulus of elasticity of the concrete slab, and the stiffness of the slab as measured by its thickness.

The curves of Fig. 8 make several points of importance clear: (1) If moment is neglected, the length of bar producing positive bearing pressure will be one-

[13] "Applied Elasticity," by S. Timoshenko and J. M. Lessels, Westinghouse Technical Night School Press, pp. 133–153.

half the length producing negative bearing pressure; and (2) for resistance to end moment alone, the length of dowel producing positive pressure is decreased 50% but the length of bar to the second point of zero deflection, or of reversed bearing, is not changed greatly. Hence, that length of bar beyond the value of $\beta x = 270°$ cannot be very effective in reducing bearing pressure at the joint or under the load. Nevertheless, since dependence is being placed upon a theoretical analysis that actually assumes an infinite length to the bar, it would be the better part of wisdom to design the dowel to project a few inches beyond the point, $\beta x = 270°$, or where $x = \frac{1.5\,\pi}{\beta}$.

*Convenient Computation of Bearing Pressure.*—Since both moment and load or shear are applied to the end of the dowel bar at the joint, the use of the length of bar up to the point, $\beta x = 0.5\,\pi$, for resistance to positive bearing pressure (with parabolic variation of bearing pressure) is not conservative. In fact, it is definitely liberal. However, it offers a logical procedure for computing dowel-bearing pressure that avoids the direct use of the factor, $K$ (a factor that varies from joint to joint and even from dowel to dowel of the same joint). Fortunately, this factor is under a fourth root radical in the formula for $\beta$ which means that the required length of the dowel bar is influenced far less directly by the sub-grade modulus than is the bearing pressure.

*The Foundation Modulus, K.*—Several persons have expressed an interest in the values of 300 000 lb per sq in. and 1 500 000 lb per sq in. that the writer has used on occasion for the factor, $K$, in Equation (1). This symbol denotes the settlement of the dowel bar, in inches, for a vertical load of 1 lb per sq in. applied to the top surface of the bar. If the dowel bar is supported on a massive foundation, the stress distribution under the bar may be somewhat as indicated

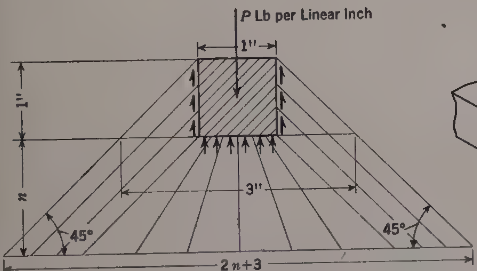


FIG. 9.—POSSIBLE PRESSURE DISTRIBUTION UNDER A DOWEL

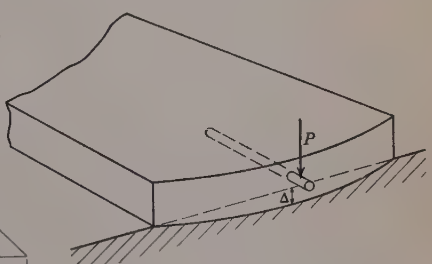


FIG. 10.—FOUNDATION MODULUS FOR A DOWEL

by Fig. 9. The average bearing pressure is less than 1 lb per sq in. even immediately under the bar because of the shears along the vertical sides of the square bar and perhaps even because of a tension bond resistance at the top of the bar. Then the bearing pressure decreases with the distance below the bar in some such manner as indicated by Fig. 9 in which it is based upon 45° lines of distribution. Hence, the average unit stress at $n$ units below the dowel bar would be,

$$f_b = \frac{1}{2\,n + 3} \qquad \text{(20)}$$

and the average vertical compressive stress over the depth, $n$, becomes:

$$f_b \text{ (average)} = \frac{1}{2}\left(\frac{1}{3} + \frac{1}{2n + 3}\right) = \frac{n + 3}{6n + 9} \quad \ldots\ldots\ldots\ldots (21)$$

For large values of $n$ this fraction becomes nearly $\frac{1}{6}$. Hence, one may write (approximately), $\delta_{1\ \text{lb}} = \frac{n}{6E}$, but,

$$K = \frac{1}{\delta_{1\ \text{lb}}} = \frac{6E}{n} \quad \ldots\ldots\ldots\ldots (22)$$

This analysis affords no definite answer to the value of $K$ for the case of a dowel bar projecting from a massive block of concrete; but it does indicate that it may be smaller than $E$. For instance, if the conditions indicated by Fig. 9 could be expected to hold for a depth of 5 ft below a 1-in. dowel bar set in a massive block of concrete, the value of $K$ would be only 10% of $E$, or about 300 000 lb per sq in. The writer doubts whether there would be a reduction of this amount but there is another factor to be considered. The tendency, both in practice and in Mr. Friberg's recommendations, is to stress the concrete rather high in bearing under the bar. For stresses above common working stresses the modulus of elasticity of the concrete itself will decrease, and plastic deformation will take place to some degree. This may result in a considerable reduction of $K$, perhaps even to 300 000 lb per sq in., when the action illustrated by Fig. 9 is also considered. Such a low value, therefore, may be the proper one to use when dowel-bar misalignment in the horizontal plane is studied. Tests are needed to verify this possibility.

Finally, there is the fact that the actual condition to be studied for the case of vertical load transfer by a dowel bar is one for which the dowel is supported by a thin slab which, in turn, rests upon a sub-grade that is expected to re-act nearly elastically for low loads. As shown by Fig. 10, there is not only a compressive deformation in the concrete under the dowel, but the slab actually curves and sinks into the foundation mat. All these factors tend to increase the deflection by a large measure and, of course, they reduce the factor, $K$.

Assume, for example, that there are 3 in. of concrete below the dowel of Fig. 10. Then, at the soil level, the load might be spread over a width of $2n + 3 = 9$ in. A width of 10 in., however, would still give only a 10 to 1 ratio of the modulus $K$ to the sub-grade modulus itself; for 100 in. the ratio would be 100 to 1. This is far from a satisfactory analysis but it may be somewhat indicative. For a sub-grade modulus of 300 lb per sq in. per in. of settlement (a rather firm material) the correspondingly indicated range of values of $K$ would be from 3 000 to 30 000 lb per sq in. Perhaps the actual value of the foundation modulus is not this low, but nevertheless it calls attention sharply to the fact that present ideas regarding dowel design may have to be revised.

Incidentally, this was the weakness in the theory that the writer had in mind in 1931 when he specified 4-ft dowels.[12] His computations on bearing pressure were made for $K$ = 300 000 lb per sq in., which was regarded as the minimum value that the foundation modulus might approach for a very thick

slab or block of concrete. For a thin slab on a soil foundation the modulus, $K$, was recognized to be subject to a probable serious reduction, and the minimum possible value of perhaps only 3 000 to 30 000 lb per sq in. dictated the specification for 4-ft dowels that was made. In a later publication, in which stresses from misalignment in the horizontal plane were being reported, a value of $K$ of 1 500 000 lb per sq in. was chosen since the purpose was to set limits on the possible disastrous effect of misalignment rather than to study average conditions. Of course, this value was used only for the study of misalignment in the horizontal plane where the soil modulus is not a factor. All this should point out the very indefinite information available regarding the foundation modulus, $K$, and the need that it should be studied in a scientific manner. The writer believes that this modulus may show a maximum variation of a hundred-fold. Only tests can answer this question.

*Longitudinal Warping Produces Severe Dowel Stresses.*—A kind of dowel stress that has been given a minimum of consideration by the author, and by all other writers, is the bending produced by longitudinal warping of the pavement. The sub-grade modulus varies from section to section, and temperature variation through the slab is not uniform. The result is that no two undoweled slabs would ever warp in identically the same manner. Hence, there are differences of elevation between the two slabs at the joint that must be equalized by the dowels. The writer has no doubt but that this difference in elevation would amount to $\frac{1}{8}$ in. in a considerable percentage of joints and $\frac{1}{4}$ in., or more, for exceptional cases of non-uniformity. To produce a settlement of $\frac{1}{16}$ in. over an average slab width of only 1 ft near the joint where the sub-grade modulus is 200 (average material) would require a dowel force of $\frac{1}{16} \times 200 \times 144 = 1\,800$ lb per lin ft of joint. (It is assumed that each slab will be displaced $\frac{1}{16}$ in. for a $\frac{1}{8}$-in. differential settlement.) The value of $\frac{1}{16}$ in. chosen is about equal to the upward curling deflection of the slab for a temperature variation of 25° F. It is important to recall that W. K. Hatt,[14] M. Am. Soc. C. E., measured an upward corner deflection of a pavement slab of $\frac{1}{4}$ in. from moisture change only. Hence, it seems reasonable to expect at least $\frac{1}{8}$ in. of differential settlement between adjacent undoweled slabs where lateral curling and longitudinal warping produced by a combination of temperature and moisture changes may readily combine with variations of sub-grade modulus to produce non-uniformity.

The writer's crude estimate of the dowel shear per linear foot of joint as that force necessary to produce a settlement of $\frac{1}{16}$ in. over a slab area 1 ft square can be refined rather easily by taking into account the elastic curve of the slab. Thus, the slab of 20-ft width and 6-in. depth may be considered to be supported on an elastic foundation and loaded at the edge with a series of vertical loads which are the dowel shears from longitudinal warping. Let $K = 200$ lb per sq in. per in. of settlement; then, by Equation (1):

$$\beta = \sqrt[4]{\frac{200 \times 240}{4 \times 3\,000\,000 \times \frac{1}{12} \times 240 \times 6^3}} = 0.031.$$

[14] "The Effect of Moisture on Concrete," by W. K. Hatt, *Public Roads*, March, 1925, p. 15.

The length of slab to the second point of zero deflection is defined by $\beta_x = 270°$ $= 1.5\,\pi$; or, $x = \dfrac{1.5\,\pi}{0.031} = 157$ in. $= 13.1$ ft. For an end deflection of $\frac{1}{16}$ in., $\delta = 0.062$ in. $= \dfrac{P - \beta\, M_0}{2\,\beta^3\, E\, I} = \dfrac{P}{2\,\beta^3\, E\, I}$ (nearly); and, $P = 0.062 \times 2 \times 0.031^3$ $\times$ 3 000 000 $\times \frac{1}{12} \times 12 \times 6^3 =$ 2 400 lb, in which $P$ is the dowel load per linear foot of joint. This value indicates that the dowel shear is approximately the load necessary to produce a settlement of $\frac{1}{16}$ in. over a slab area, 1 ft wide by 1 ft 4 in. long, for the particular case studied.

*Severe Combinations of Dowel Stress.*—In order to evaluate the author's conclusions, it is necessary to determine the possible severity of load combinations. This study will include: (1) A load transfer of 50% of the author's 4 000-lb rear wheel load (which is a smaller load than is permitted in many States); (2) longitudinal warping producing a differential settlement of $\frac{1}{8}$ in. between adjacent undoweled slabs; and (3) misalignment of $\frac{1}{8}$ in. per ft, or $\frac{1}{4}$ in. for a standard length dowel. Calculations will be made for three $K$-values: A value of 300 000 lb per sq in. as an upper limit with a soft sub-grade; an intermediate value of 30 000 lb per sq in.; and a minimum $K$-value of 3 000 lb per sq in. Dowels are $\frac{3}{4}$ in. at 12-in. spacing. For $K =$ 300 000, Equation (1) yields $\beta = 0.59$. The required length of dowel would be $2\,(1.5 \times \pi) \div 0.59$ $= 16$ in. For $K =$ 30 000, $\beta = 0.33$. The corresponding length of dowel would be $2\,(1.5 \times \pi) \div 0.33 = 29$ in. Finally, for $K =$ 3 000, $\beta = 0.187$. The dowel length then would be $2\,(1.5 \times \pi) \div 0.187 = 50$ in. Thus, there is the possibility of 4-ft dowels being required, but as a more probable case which is close to present-day practice a study will be made of a dowel of 30-in. length.

The minimum possible bearing stress for a $\frac{3}{4}$-in. dowel, 30 in. long, will occur when positive pressure exists over one-third the length of 10 in. For parabolic variation of pressure and a load transfer of 2 000 lb, the bearing stress is $\dfrac{3 \times 2\,000}{0.75 \times 10} = 800$ lb per sq in. The corresponding bearing stress for longitudinal warping would be $\dfrac{3 \times 2\,400}{0.75 \times 10} = 960$ lb per sq in. To this total of 1 760 lb per sq in., which is already as high as any one is willing to recommend, must be added according to the author's estimate another 2 000 lb per sq in. for misalignment, a total of 3 760 lb per sq in. even with 30-in. dowels. Satisfactory functioning of much shorter dowels during the full life of the pavement is obviously questionable.

*Conclusion.*—The author has presented an interesting study of an old problem. The problem has received less consideration than it deserves because expansion joints, once placed, are not open to inspection until the road is torn up. Failure of road-slabs at other places than the expansion joints have been so frequent that dowel failures have received scant study and little comment. Nevertheless, the crudest possible calculations show rather conclusively that short dowels, widely spaced, are of little value for load transfer and that they subject the concrete to high bearing stresses which reduce, rather early in the life of the pavement, whatever initial effectiveness the dowels may have had.

The author's conclusions are reasonable except in regard to the lengths of dowels. His interpretation of the significance of the factor, $K$, which must combine in its make-up the sub-grade modulus, the modulus of elasticity of the concrete, and the slab stiffness as measured by its thickness led him to overvalue the significance of his calculations in regard to dowel length. No one can guarantee that the value of $K$ for vertical loading may not be very much lower than the value of 300 000 used by the writer in his first studies of this problem, whereas the author's conclusions are based upon a value of 1 000 000 for $K$. The result of using a low foundation modulus in the author's calculations might be to revise his conclusions that 24-in. dowels may be shortened to 12 in. and lead to the conclusion that standard 24-in. dowels should be lengthened instead. No one can answer this criticism until adequate tests have been made to set limits upon the foundation modulus, $K$.

During the interim the only reasonable procedure in analysis is to study each design feature for the corresponding critical value of the sub-grade modulus. Thus, bearing pressure, which controls the diameter and spacing of dowels, must be studied for a relatively high value of the foundation modulus. The dowel length, on the other hand, will be controlled by a relatively low value of the foundation modulus. Even when tests have established the unknown relationships involved, it will still be proper to follow this procedure because the foundation modulus will never be constant for any reasonable length of roadway.

AMERICAN SOCIETY OF CIVIL ENGINEERS

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DISCUSSIONS

# LAND SURVEYS AND TITLES

## FIRST PROGRESS REPORT OF THE JOINT COMMITTEE OF THE REAL PROPERTY DIVISION, AMERICAN BAR ASSOCIATION AND THE SURVEYING AND MAPPING DIVISION, AMERICAN SOCIETY OF CIVIL ENGINEERS

### Discussion

BY MESSRS. JOSEPH C. FEDERICK AND WAYNE D. HEYDECKER, AND EARL F. CHURCH

JOSEPH C. FEDERICK,[2] ESQ., AND WAYNE D. HEYDECKER,[3] ESQ. (by letter).[3a] Although many States utilize the State plane co-ordinate systems developed by the U. S. Coast and Geodetic Survey, only three have given legal recognition to the systems—New Jersey in 1935, Pennsylvania in 1937, and New York in 1938. The New York law becomes effective July 1, 1939. The Joint Committee recommendation that each State Legislature enact an enabling law for the establishment of State co-ordinate systems should be followed by State and local societies who should present to the appropriate authorities in each State the advantages to be derived from the adoption of such a system.

The half-mile limitation, specified in Section 6 of the Appendix, is a critical point. Such a restriction, of course, is intended to insure a uniformly high grade of surveying for cadastral purposes. However, too severe a limitation without some method of providing flexibility may prevent the co-ordinate systems from coming into wide-spread use.

This section evoked considerable discussion on the Advisory Committee on Maps and Surveys appointed by the New York State Planning Council. This Advisory Committee, which prepared the New York Act in co-operation with

NOTE.—This First Progress Report of the Joint Committee of the Real Property Division, American Bar Association, and the Surveying and Mapping Division, American Society of Civil Engineers, on Land Surveys and Titles, was published in November, 1938, *Proceedings*. Discussion on this report has appeared in *Proceedings*, as follows: January, 1939, by William Bowie, M. Am. Soc. C. E.

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[3a] Received by the Secretary December 21, 1938.

State and Federal authorities, gave consideration to Section 6 as suggested in the Model Act and the following three alternative proposals:

> *Alternative No. 1.*—"No coordinates based on the New York State Coordinate System purporting to define the position of a point on a land boundary shall be presented to be recorded in public land records or deed records unless such point is within one-half mile of a triangulation or traverse station established as prescribed in Section 5 of this Act, unless the State department authorized to administer this Act shall by its rules and regulations increase or decrease such one half mile distance for the whole State or any area or areas thereof."
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> *Alternative No. 2.*—"No coordinates based on the New York State Coordinate System, purporting to define the position of a point on a land boundary, shall be presented to be recorded in public land records or deed records, unless such survey is begun at a control point and carried through and ended at a second control point, and the coordinates so placed shall check out on the second point within the requirements of the accuracy and precision such as shall be prescribed for the area affected."
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> *Alternative No. 3.*—"No coordinates based on the New York State Coordinate System, purporting to define the position of a point on a land boundary shall be presented to be recorded in public land records or deed records, unless such point is within one-half mile of a triangulation or traverse station established as prescribed in Section 5 of this Act, or unless appropriate geodetic and projection methods have been applied in the establishment of such coordinates and the degree of precision and accuracy shall conform to the standards laid down by the U. S. Coast and Geodetic Survey for second order ground traverse, and a mean error of closure of 1 in 10,000 ft."

Alternative No. 1 is based on the phraseology of the Pennsylvania Act which delegates to a State authority the power to increase or decrease such half-mile limitation. This section in the New York Act as adopted is quite similar to the Pennsylvania requirements and reads as follows:

> "No coordinates based on the New York Coordinate System purporting to define the position of a point on a land boundary shall be presented to be recorded in public land records or deed records unless a point in the survey is within one-half mile of a triangulation or traverse station established as prescribed in section four of this act, or unless the department of public works which is the agency hereby authorized to administer this act shall by rules and regulations increase or decrease such one-half mile distance for the whole state or any area or areas thereof."

The suggestion of the Joint Committee, that a bureau of maps and surveys be established in each State to administer the co-ordinate act, is a logical development. The functions of such an agency could properly and advantageously go beyond the administration of a co-ordinate act. The agency should also serve as a central clearing house where engineers, surveyors, or the general public might ascertain the availability of maps of the State and its political subdivisions.

The dissemination of control-survey data is also of considerable importance. Usually such information is supplied upon request only. Little is done to promote or facilitate the utilization of available information. As a matter of regular administrative procedure, the suggested bureaus of maps and surveys

might file all co-ordinate data pertaining to the respective unit with an appropriate agency in each level of government. For example, the data pertaining to an entire county could be filed with the county engineer, the town data with the town superintendent of highways, and the city and village data with the engineer for the community.

Earl F. Church,[4] Assoc. M. Am. Soc. C. E. (by letter).[4a]—The geodetic triangulation of the U. S. Coast and Geodetic Survey affords an excellent basis for a nation-wide system of cadastral surveys upon a single datum. Breaking this geodetic system down by States into individual rectangular co-ordinate systems, based upon map projections, does not in any way destroy the unity of all the surveys of all of the different States, as far as the geodetic datum is concerned; yet these rectangular co-ordinate systems offer a practical means for every land surveyor to utilize geodetic control.

The initial report of the Joint Committee recognizes the importance of placing all property surveys and descriptions within each State upon such a co-ordinate system. As a matter of fact, the importance of this step in modern cadastral surveying practice can scarcely be exaggerated or over-estimated.

In States where the rectangular co-ordinate system, based upon the Lambert or Transverse Mercator projection, has been adopted officially for property surveys, a coherent organization of the work of extending the Federal geodetic control, of course, becomes necessary. The control survey of an entire State should be under the direction of a competent geodesist. However, the organization of the work by counties, with a very brief training of the field and office personnel, would be practical and inexpensive and would insure results of the required precision. There is no reason why county organizations could not provide, in a very brief time, all the permanent control points needed for a continuous State-wide system of property surveys, all based upon geodetic datum. The practicability and the feasibility of doing this necessary control work is mentioned merely to justify the Joint Committee's recommendation that a standard rectangular co-ordinate system, based upon geodetic datum, be adopted for all property surveys within each State.

The details regarding the organization of this surveying within individual States will doubtless be considered in subsequent reports of the Joint Committee. Certainly the first progress report recommends the logical initial step for each State, namely, the legal adoption of an official rectangular co-ordinate system based upon standard geodetic datum and an official map projection, for all property surveys and property descriptions.

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[4a] Received by the Secretary February 8, 1939.

AMERICAN SOCIETY OF CIVIL ENGINEERS

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# DISCUSSIONS

# STATE-WIDE SURVEYING PRACTICE IN MASSACHUSETTS A SYMPOSIUM

## Discussion

BY MESSRS. LLOYD G. FROST, WILLIAM BOWIE, AND R. M. WILSON

LLOYD G. FROST,[12] ASSOC. M. AM. SOC. C. E. (by letter).[12a]—The splendid accomplishments of the Massachusetts Geodetic Survey, so ably set forth by the author, cannot be over-rated in their value to the Commonwealth of Massachusetts and to the long-neglected field of surveying and mapping, as well as to the profession at large. Mr. Houdlette has so clearly depicted the conditions that exist generally in the land-surveying field and the relation of a system of rectangular co-ordinates thereto that every engineer and surveyor, in the light of his own experience, will readily recognize the indisputable superiority of the co-ordinate method and look hopefully for extension of such State systems on a national scale.

*The Louisiana System of Plane Co-ordinates.*—The Louisiana Geodetic Survey was established at the same time as the Massachusetts project and under like conditions. The same vicissitudes were met that visited themselves on the latter, and, in general, the details of organization and field procedure are parallel. Mr. Houdlette's description of the projection frame applies to that for Louisiana, except that the configuration of the State made two projections necessary—North and South. As with Massachusetts, the U. S. Coast and Geodetic Survey selected the projections and prepared the necessary tables of data for computations.

The difficulties encountered in securing competent personnel to perform the work adequately may have been somewhat greater in Louisiana because of the much larger area of the State, but the various obstacles were surmounted surprisingly well in the earlier stages of organization. The standards of accuracy maintained for the Louisiana project differ from those in Massachusetts and

NOTE.—This Symposium was presented at the meeting of the Surveying and Mapping Division, Boston, Mass., October 7, 1937, and published in November, 1938, *Proceedings*. Discussion on this paper has appeared in *Proceedings*, as follows: January, 1939, by Messrs. H. J. Shea, and Philip Kissam.

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[12a] Received by the Secretary December 9, 1938.

New Jersey,[13] as indicated by the following comparison (in which $S$ is the distance run, in miles):

| | Traverse | Levels |
|---|---|---|
| Louisiana | 1 : 20 000 | $0.25\ \sqrt{S}$ |
| Massachusetts | 1 : 10 000 | $0.4\ \sqrt{S}$ |
| New Jersey | 1 : 10 000 | $0.3\ \sqrt{S}$ |

The interval between stations in the Louisiana network varied from 0.5 mile to 1 mile during the early years of the program. Experience showed that in many cases these distances were too great to permit maximum availability to the local user, and the optimum interval was set at 0.25 mile except in certain unusual cases. Wherever time and facilities permit, longer tangents are being broken down to this standard.

The strength of field forces in Louisiana varied from an average of fifteen parties to a maximum of thirty during a short period in 1936–1937. In February, 1938, a reduction was made to ten parties in a reorganization effected to increase efficiency.

The work accomplished by the Survey, through September, 1938, in comparison with that given by Mr. Houdlette for Massachusetts is as follows:

| | |
|---|---|
| Traverse: | |
| Extent, in miles | 5 964.82 |
| Number of stations established | 12 234 |
| Miles of traverse with preliminary co-ordinates | 531 |
| Miles of traverse with final co-ordinates | 3 754.72 |
| Average closure error | 1 : 35 000 |
| Bench-Marks: | |
| Extent of second-order levels, in miles | 5 964.82 |
| Number of bench-marks | 14 681 |

Protracted study of the system of field procedure led to the conviction in February, 1938, that maximum efficiency was not being attained. The field organization, unvaried since the beginning of the project, was as follows at that time:

Each party consisted of ten men—a chief of party, transitman, levelman, and seven rodmen and chainmen; transportation was by owner-operated truck, the driver working as chainman; and each party was provided with one transit and one level.

The methods of geodetic surveying necessary to insure the required standards of accuracy would not permit doing the various operations simultaneously with this number of men. There appeared to be a considerable waste of time that was unavoidable under the conditions. At no time since the inception of the project did the records of work performed indicate a rate of progress consistent with the number of men engaged on the work. The available funds did not permit any appreciable changes at that time, but it was decided to institute some investigations and experiments in anticipation of a new appropriation.

[13] *Bulletin No. 46*, New Jersey State Dept. of Conservation, 1938.

Two parties were chosen as "guinea pigs," and the personnel increased to seventeen men, classified as follows: Observation section—chief of party, recorder, and two rodmen; measurement section—transitman, recorder, two chainmen, and three rodmen; level section—levelman, and two rodmen; and monumenting section—truckman, and two laborers. It was believed that a party so organized could carry on the several operations simultaneously with a maximum of efficiency and accuracy. The results obtained more than realized expectations and all parties operating in the State (fifteen in number) were reorganized on this basis. Two division engineers were appointed with headquarters in the northern and southern parts of the State and intensive supervision of field operations instituted.

Each party was supplied with two transits and other necessary equipment to conduct the work with a minimum of lost motion and duplication of effort. Indications are that these parties, comprising a total of 255 men, will perform considerably more work than thirty parties of ten men each. The proportionate saving in cost is greater than indicated by the smaller personnel, due to reductions in the higher brackets of salaries, and in transportation costs.

Most important of all is the time factor—experience thus far (1938) leads to the conclusion that it will be reduced by one-half.

No triangulation has been attempted by the Louisiana Geodetic Survey. The 1938 winter program of the U. S. Coast and Geodetic Survey contemplates completing the Louisiana network. One party has been engaged in first-order traverse for about one year.

*Publication of Geodetic Data.*—A constantly increasing demand for station descriptions created such a burden of clerical work that it became necessary to establish some method of placing the accumulated data at the disposal of users in a compact and accessible form.

After a study of available means of distribution, it was decided to issue a series of geodetic quadrangles, enclosing 0° 15′ of latitude and longitude, identical in geographic position, nomenclature, and scale with those of the U. S. Corps of Engineers and the U. S. Geological Survey. A total of 208 such quadrangles was required to cover the State and, in addition, a series of three-minute quadrangles (6 in. to the mile), covering the more important municipal areas, will be published.

Quadrangles are printed in black and white with all water in blue. All control points are located and identified and sufficient topographic information, together with subdivision lines of the rectangular land office surveys, are shown to insure facility in selecting control points for any purpose. The reverse of each quadrangle bears complete geodetic data for each station.

The location of rectangular survey lines is from original township plats, adjusted to field ties made by the U. S. Geological Survey, the State Board of Engineers, the U. S. Corps of Engineers, and the Louisiana Geodetic Survey, and topography from controlled aerial survey data is from the same sources. In addition to the quadrangles an index or key map is published, bearing on its reverse side a brief explanation of the method of using the data. Mimeographed descriptions of reference marks are available with each quadrangle. The user thus has at hand a graphic portrayal of the available control points in

the section where he proposes to work, together with ties to facilitate locating them in the field, and the data necessary for their use.

*Land Ties.*—The original rectangular surveys in Louisiana date back as far as 1806. Prior to this time innumerable grants had been made and surveyed under Spanish and French rule. Where the "vara" obtained in early Texas surveys, the "arpent" was the unit of measure used in Louisiana prior to the United States Land Office subdivision.


FIG. 6

Because of the type of instruments available, and other factors, familiar to every engineer, that influenced the making of early surveys, the conditions existing in Louisiana with reference to identifiable land corners and markers were confusing and discouraging until the present co-ordinate system was established.

Wherever it is possible to locate and identify original corners authentically, they are tied to the control network. The special marker shown in Fig. 6(*c*) is used, and at a satisfactory distance (usually between 400 ft and 500 ft) a

reference marker (Fig. 6(*d*)) is set. The co-ordinates of all such corners are included with the data on the corresponding quadrangles, together with the azimuth of the reference point. This enables the engineer or surveyor to start from any such corner with a known position and a true bearing.

Many amusing (and at times appalling) discrepancies are encountered in adjusting the original surveys to actual positions determined by geodetic methods. There have been instances, furthermore, in which the work of the early surveyor stood up amazingly well. Within the life of the Survey, all original corners that can be located will be thus included in the original network.

*State Boundaries.*—That part of the boundary between the Republic of Texas and Louisiana north of, and not formed by, the Sabine River, was surveyed prior to 1845. Earthen mounds about 5 ft high and 15 ft in diameter were established at intervals of 1 mile. A traverse of this boundary has been run by the Louisiana Geodetic Survey, and monuments bearing the names of the Governor of Texas or of Louisiana on traverse markers (Fig. 6(*b*)) are established in all existing mounds.

That there were markers used other than mounds is evidenced by the discovery of a marble monument, 12 in. by 12 in., projecting well above the ground, and marked on the east "Louisiana" and on the west "Republic of Texas." The location of this boundary by plane co-ordinates is important not only as a State line, but as a line of demarcation across the rectangular subdivisions, and great care is being observed to locate and connect all land corners near-by.

The northern boundary of the State was included in this traverse. Although it was well monumented, there were no available data on geographic location of these points and their connection to the control network was important and necessary. That part of the northern boundary east of the Mississippi River is controlled as a line of latitude and it will be run and monumented as such.

*Legal Recognition.*—The New Jersey Legislature has enacted legislation establishing the plane co-ordinate system in that State as the base of all surveys. Such an act was for presentation to the 1939 session of the Louisiana Legislature, incorporating provisions to guarantee work ability and penalties for non-observance. The legislature of 1918 enacted legislation which is applicable to field operations of the Louisiana Geodetic Survey in granting rights of entry and affording protection of monuments and structures.

*Use of Information.*—In Massachusetts, New Jersey, and elsewhere keen appreciation has been evidenced of the vital importance of making all accumulated data easily accessible so that its application can be made universal. In his conclusion Mr. Houdlette states that in Massachusetts the control stations are located on individual town maps, together with available data. It is the writer's understanding that the Commonwealth of Massachusetts is divided into about 300 "towns"; Louisiana, with its much greater area, is divided into 64 "Parishes." Plotting of control data to a suitable scale on Parish maps would have resulted in such maps being too cumbersome for convenient use, irregular in size and bearing no definite relation to other compilations of related data.

The quadrangle system, as adopted, appears to be loosely comparable to the Massachusetts town system, except in one particular, which is of importance in Louisiana. Exploration for oil and minerals has extended into all parts of the State, as well as large-scale surveys for other purposes. In nearly all these operations, engineers are dependent upon the quadrangles of the U. S. Engineer Corps and the U. S. Geological Survey for topographical and cultural data, and upon the control of the U. S. Coast and Geodetic Survey and the Louisiana Geodetic Survey. The compilation of geodetic information upon identical quadrangles achieves a uniformity in size, location, scale, and area of base maps that is of considerable value to the user.

A "consummation devoutly to be wished" is the publication, by every State, of a uniform series of maps bearing geodetic information based on plane coordinate systems such as Mr. Houdlette describes, the whole to form the base for a series of topographic and cadastral maps on a nation-wide scale.

To this end Mr. Houdlette's paper, in recounting the accomplishments of Massachusetts, may prove to be of no small value in arousing realization of the need for such a program and in helping to point the way.

The Louisiana Geodetic Survey is sponsored by the Louisiana Highway Commission (Harry B. Henderlite, M. Am. Soc. C. E., State Highway Engineer) with the assistance and co-operation of the State Department of Conservation, the State Board of Engineers, and the Orleans Levee Board of the City of New Orleans, La.

William Bowie,[14] M. Am. Soc. C. E. (by letter).[14a]—The story told by Mr. Houdlette should be given thoughtful consideration by all engineers and other users of land, and by the officials of States, counties, and cities, whose duties include the advancement of the welfare of their citizens. After many years of effort on the part of a few engineers and planners to arouse the governing bodies of the United States to the desperate mapping and surveying situation, at last something is being done about it in at least a few States and political subdivisions.

The opportunity came when the Administration, late in 1933, sought work that could be done by engineers and others who were unemployed. Geodetic surveys were suggested as professional projects that would provide jobs for thousands of idle engineers. Work was organized in all of the States by the officials of the U. S. Coast and Geodetic Survey, with a local director in each State. Within less than two months after the start, more than 10 000 persons had been employed, and of these, more than 8 000 were engineers and other college and university men and women. The project was a Federal one. The State directors were given wide authority as to who should be employed, and were authorized to discharge any one who could not, or would not, perform services that met their requirements. After a few months the project ceased to be under the Federal jurisdiction, and was turned over to the States, cities, and counties. It is to the lasting credit of those in charge of the subsequent projects that they carried on in the face of great difficulties and discourage-

[14] Former Chf., Div. of Geodesy, U. S. Coast and Geodetic Survey, Washington, D. C.
[14a] Received by the Secretary December 16, 1938.

ments. In several States and counties comprehensive geodetic surveys have been made. Massachusetts is one of the outstanding ones.

The U. S. Coast and Geodetic Survey prescribed specifications for the geodetic surveys that would give to the results an accuracy needed for all except the most accurate surveying that might be based on them. It was required that the error of closing of a line of traverse should not be greater than $\frac{1}{10\,000}$. This is a greater accuracy than is obtained on most local alignment surveys made for construction and land boundary. The emergency geodetic surveys proved that this accuracy could be obtained with the equipment in common use by engineers. In Massachusetts Mr. Houdlette required a greater accuracy than that prescribed by the U. S. Coast and Geodetic Survey. This led to his securing an average closing error of only $\frac{1}{35\,400}$ for his traverses. The extra effort expended in improving the closures was more than justified. The work will not have to be repeated in the future, and the results will meet the most exacting needs of the engineer and surveyor.

The minimum limit of closure was set at $\frac{1}{10\,000}$ because it was felt that in some States it would be difficult to find engineers who could begin immediately to produce surveys of higher accuracy. Even the $\frac{1}{10\,000}$ survey was a severe problem for some of the directors. One of them reported that he could not find a single engineer in his State, eligible for relief employment, who could run a traverse with the required accuracy. A few men had to be trained first to do the work in a satisfactory manner, and then used to train others. The writer cites this to show to what a low ebb the surveying of this country had fallen. Several millions of dollars have been spent on the relief geodetic surveys and valuable results have been secured in several States. The unit costs have been higher than if the work had been done under normal employment conditions, but the educational value alone of the emergency geodetic surveys more than justified the expenditure.

It cannot be expected that the States and their political subdivisions will continue indefinitely to receive relief funds for making geodetic surveys; but those surveys should be continued until there is a traverse or triangulation station within easy reach of the engineer or surveyor who may wish to have his work co-ordinated with surveys made by others in the same region. This means that in each State there should be an agency charged with the duties of making geodetic surveys, and furnishing information to those needing it. Such an agency should have authority to set minimum limits of accuracy for all agencies of the State Government that make surveys for engineering operations, and for the location or re-running of the boundaries of public and private property. If a few traverse parties in a State should operate continuously for five or ten years much could be accomplished. So many uses would be found for the results that there would be no difficulty in finding support for the completion of the work for the State.

One of the urgent needs of most of the States is the establishment of Land Courts. In a most interesting manner, Mr. Humphrey has described the Court dealing with title to land in Massachusetts, and the surveying that is done for the Court. Each State may well create a Land Court after the Massachusetts pattern. It would be a boon to engineers who have as their primary function the placing of valuable structures on the land. They should be sure that they have the boundaries accurately defined and located before starting their work.

Massachusetts is to be commended and congratulated for having sensed the importance of good land surveys, and for giving the effort necessary to make such work effective.

In recent years, the Society has published a number of papers that have told of the chaotic condition of land boundary surveys in such States as North Carolina,[15] Florida,[16] Massachusetts,[17] and Texas.[18] No doubt the conditions they portray are the same as in the other States. The writer knows of no State in which satisfactory surveys have been made on any comprehensive plan.

Mr. Houdlette has justly stressed the difficulty of using the national triangulation net as the base for local surveys on the system of spherical co-ordinates; but that difficulty has been overcome by devising State Plane Co-ordinate Systems. The engineer engaged on local surveys can use the plane co-ordinates of the triangulation and traverse stations, and needs to pay no attention to the computations that are involved in the transfer of spherical to plane co-ordinates. That work is done for him by the mathematicians of the U. S. Coast and Geodetic Survey or agencies like the Geodetic Survey of Massachusetts. There is really no excuse on the part of responsible officials of a State for delaying the adoption of plane co-ordinates, and the private engineer is doing himself a disservice if he does not use them.

There is now widespread sentiment in favor of better survey methods, and this sentiment will soon be translated into action. One of the first steps to be taken to bring better conditions into being should be the completion of the National Triangulation net. In the past, the triangulation has been considered by some as merely the frame for the topographic mapping; but the triangulation has so many other uses that it should be finished as soon as possible and not merely meet the demands of the topographic engineers of the Federal Government. The plan for the triangulation calls for the spacing of the arcs of triangulation at intervals of about 25 miles. This is the immediate objective, but the ultimate one is that eventually there will be no place east of the Rocky Mountains more than 6 miles from a station, whereas this distance is increased to 12 miles in the mountainous regions. This spacing is considered as desirable for the control of the maps that will be compiled from photographs taken from the air. Such spacing of the triangulation stations will make it possible for the engineers of the State Governments to have traverses run between pairs of stations, along railroads and highways, for the purpose of setting many stations along the routes for use by engineers and surveyors engaged on local surveying for engineering and land boundary work.

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[15] *Civil Engineering*, January, 1937, p. 33.

[16] *Loc. cit.*, July, 1938, p. 451; June, 1938, p. 386; and May, 1938, p. 331.

[17] *Loc. cit.*, December, 1937, p. 824.

[18] *Loc. cit.*, November, 1938, p. 722; and July, 1937, p. 507.

Perhaps no one is to blame for the slip-shod surveying that has been done in the United States during the past hundred years, but the bad practice should not be continued. The Corps of Engineers of the U. S. Army has been making surveys for river and harbor improvement and flood protection for many years, and, although the surveys have met their immediate needs, in most cases they were of a low order of accuracy, and in most instances no permanent monuments were left on the ground. This triangulation or traverse cannot add much to the national horizontal control system of the country.

The Topographic Branch of the U. S. Geological Survey has done an enormous amount of control surveying for its topographic mapping; yet only a very small percentage of this has been of such a standard as to make the results of value to the engineer or surveyor who is making surveys for engineering work and land boundaries. In their old work they left few or no permanent monuments. They should not be blamed for doing work with no more accuracy than was just sufficient for their small-scale mapping. They received a pitifully small sum of money for their mapping, so they could not divert much of it to the control surveying. Since there are so many needs for the results of the control surveys, however, the Geological Survey should strengthen its work in this respect. This applies also to the control surveys made by the Corps of Engineers.

If the objectives outlined by Messrs. Houdlette and Humphrey are to be reached in each of the States, all State agencies, such as the Highway Department, should make their control surveys with the accuracy needed to meet future use of the data secured. To do this would cost the State very little additional. The first step would be to purchase good instruments. The day of the 1-min transit, the non-standardized tape, the old-fashioned level, and the wooden leveling rod, has passed just as the day of the wheelbarrow and the ox cart has passed. To continue to use the old types of instruments is bad practice and wasteful. Either poor results are obtained, or an undue length of time is taken to secure accurate results. In every branch of engineering except surveying and mapping the best available instruments, machines, and equipment are bought and used. It is costing money to continue the use of surveying instruments that were scarcely up to date even two generations ago.

Although standards of surveying have been low, there is no cause to be pessimistic over the mapping and surveying situation. There are dozens of engineers and surveyors to-day who are making strong efforts to put better methods into practice, whereas until quite recently there were few who even knew there were problems to be solved. This Symposium is an indication of awakening interest in about the oldest and the most neglected branch of engineering.

R. M. Wilson,[19] M. Am. Soc. C. E. (by letter).[19a]—One of the successful projects that grew from seeds sown in 1933, in each of the 48 States, by the U. S. Coast and Geodetic Survey under an appropriation by the former U. S. Civil Works Administration, is described by Mr. Houdlette. In several

[19] Chf., Section of Computing, U. S. Geological Survey, Washington, D. C.

[19a] Received by the Secretary January 18, 1939.

States these seeds fell upon fertile soil, as they did in Massachusetts. There they sprouted and were carefully cultivated, even without the continued support of funds from the Civil Works Administration, and they have lived to bear valuable fruit.

The Massachusetts Geodetic Survey is a credit to those who have supported and administered it not only because of its success as a relief project but also because of the quantity and high quality of the work done. The supplementary control surveys that have been completed will surely repay the State of Massachusetts for its support of the project. The uses of the new geodetic control in connection with cadastral surveys and land titles have been described by both authors of the Symposium; much of the valuable harvest that the State will reap will be through these uses.

At present (1939) there is in progress, also, a program of re-mapping by the Geological Survey of the United States Department of the Interior in co-operation with the State of Massachusetts for the purpose of making, by modern methods, up-to-date topographic maps of the State. Control surveys are needed for this new mapping, and ordinarily it would be necessary for the Geological Survey to spend considerable sums of money for the required preliminary surveys before the actual mapping could begin; but with the results of adequate control surveys made by the Massachusetts Geodetic Survey already available, a larger proportion of the funds appropriated for mapping can be applied toward the primary purpose of the program. In this way, also, the State is reaping a genuine harvest because it cultivated the project that has become the Massachusetts Geodetic Survey.

In his historical sketch of early surveys in Massachusetts, Mr. Houdlette makes a statement that may be misleading to those who are not familiar with the facts. In his reference to the mapping authorized in 1884, he conveys the impression that the co-operative agreement was between the State and the U. S. Coast and Geodetic Survey, then a bureau of the U. S. Treasury Department; but Chapter 72 of the Acts of 1884 of the Massachusetts Legislature provided for the mapping of the State in co-operation with the U. S. Geological Survey, not the U. S. Coast and Geodetic Survey.

That agreement, made in 1884, produced the original topographic "quadrangle" maps of Massachusetts which were published by the Geological Survey. They are known to all map users in the State, and their popularity is attested by the large number of copies that have been distributed since they first became available, shortly after the completion of the field surveys in 1888.

The co-operative agreement, defined by Chapter 72 of the Acts, is an important one in the history of the U. S. Geological Survey. It was the first of the agreements of its kind made with States wishing to expedite topographic mapping within their borders. The success of the program established it as a model for co-operative agreements with other States, and many similar agreements have since been made. At present the U. S. Geological Survey has co-operative agreements for the making of topographic maps with 17 States and with Puerto Rico. One of these has resulted in the new co-operative mapping program previously mentioned.

The new program in Massachusetts has been undertaken because the existing maps on a scale of about 1 mile to the inch, which were made between 1884 and 1888, do not meet the exacting modern requirements. The new maps, published on larger scales with smaller contour intervals, permit the showing of greater detail and are up to date in showing developments in the works of Man. Thus, they are more nearly consistent with the present economic importance of the areas they represent. The 192 separate sheets, whole and fractional, that will be required to show the entire State, are first reproduced as lithographs on a scale of 1 : 24 000. To the present time, 51 have been reproduced as unedited advance sheets in two colors. Beginning now (1939), however, the advance sheets will be edited and reproduced as three-color lithographs. The final publications are engraved maps on a scale of 1 : 31 680.

AMERICAN SOCIETY OF CIVIL ENGINEERS

Founded November 5, 1852

DISCUSSIONS

# THE YELLOW RIVER PROBLEM

## Discussion

By Messrs. J. W. Beardsley, and Elliott J. Dent

J. W. Beardsley,[14] M. Am. Soc. C. E. (by letter).[14a]—Exceptionally detailed and comprehensive data on a difficult problem are presented in this paper. To appreciate the work involved one must consider the Chinese language which is not adapted to technical terms, the lack of co-operation between adjacent provincial authorities during past centuries, and the failure of the National Government to collect data and to plan and construct protective works.

During 1918–1920, the Grand Canal Improvement Board collected samples of silt in the vicinity of Shihlipu and the railroad crossing near Tsinan Fu by means of a hand pump and a pipe which could be lowered to any desired depth. A silt load of 15% by weight was rather high. Samples were also collected from old silt beds and measured carefully. It was concluded that 15% by weight averaged close to 10% by volume. Recently deposited silt beds, sun dried, were cracked into irregular blocks about 2 or 3 ft wide and 3 or 4 ft long. The cracks were 1 ft to 2 ft deep. Along the edge of such banks, where the current had been relatively swift, were generally found small areas of fine sand suitable for use in an hour-glass. Probably such sand would be harmful to soil fertility, whereas the average run of the silt would be beneficial.

The deltaic central plains are said to have been very fertile many years ago; but the intense cultivation during past decades has materially reduced fertility. Missionaries reported that their census of areas adjacent to Tsining, on the Grand Canal in southwestern Shantung, gave a population of about 2 000 per sq mile. A food crop is a prime necessity. The farmers live mainly in the protected villages. A common type of irrigation in this area is by water drawn from wells which have a stone post on each side suitable for using a windlass. The farmer carries the windlass and bucket from the village, adjusts the windlass in the stone posts, draws a bucket of water, and carries it to his small cultivated plat.

Note.—This paper by O. J. Todd, M. Am. Soc. C. E., and S. Eliassen, Assoc. M. Am. Soc. C. E., was published in December, 1938, *Proceedings*. This discussion is printed in *Proceedings*, in order that the views expressed may be brought before all members for further discussion of the paper.

[14] Cons. Engr., Syracuse, N. Y.

[14a] Received by the Secretary January 31, 1939.

This part of China has no "April showers to bring May flowers." It has slight snowfall or rain during the spring. Soil moisture is not sufficient to sprout early plantings. The later seasonal rains usually interfere with the harvest.

Data are not available relative to the decreasing fertility of these deltaic areas. Sir William Willcoks referring to the displacement of "basin" irrigation in Egypt by the then new perennial irrigation said:[15] "The rich muddy water of the Nile has been the mainstay of Egypt for many generations, it can no more be dispensed with today than it could be in the past."

The Nile and the Yellow rivers have many similarities. Both are deltaic rivers. Both lack tributaries of any importance several hundreds of miles above their mouths. Both have built up silt banks higher than adjacent lands. A notable dissimilarity is that the Nile has been orderly. It is the life of Egypt. The Yellow River is an unruly bandit to such an extent that it is called "China's sorrow."

The authors' discussion of "Irrigation" and "Fertilization by Flooding" are exceptionally interesting. Similar comments on reclamation would have been of equal interest. With considerable hesitation, the writer ventures to make two suggestions for making a useful servant, at least a part of the time, out of the unruly Yellow River:

(1) Adjacent to the river's old bed prior to 1851 (and in several other localities near its present bed) are a number of shallow lakes and marshy areas too wet for cultivation. Is it practicable to fill such areas with the July floods? The silt loads in the vicinity of the Peiping-Hankow Railway Bridge frequently carry 15% or more of silt by weight, or 10% by volume when compacted. An impounded basin having a maximum depth of 5 ft would deposit a half foot of soil within a few days. Within a few years such areas could be converted into "culturable" land.

Non-fertile sandy areas could be made productive by impounding a depth of a few feet of flood waters, heavy with silt, for two or three days before draining. If such an operation could be repeated annually for a few years, high fertility would be maintained for many following years.

(2) Is it practicable to flood those areas near the river, where irrigation is difficult, with the low-water flow of late winter or early spring? Some soils take up moisture slowly and the retention of the water might be necessary for a week or more before sufficient moisture is absorbed to sprout early plantings. The basin must then be drained and the retained moisture should be sufficient to carry the crop until the arrival of the summer rains.

The silt load is then generally about 1 per cent. If a depth of 2 ft were impounded, the silt deposited would be of considerable value as a fertilizer, especially if the operation can be repeated each year. This plan conforms to the "basin" irrigation of ancient Egypt.

As long as a stable government control is disturbed and co-operative maintenance of the system of dikes remains impracticable, the danger that the

[15] "The White Nile and The Cotton Crop (No. 2)," by Sir William Willcoks; a lecture delivered at a meeting of the Khedivial Geographical Society, January 25, 1908.

Yellow River will usurp an unpredictable channel to the sea is increased manyfold. Sooner or later these river problems must be solved.

ELLIOTT J. DENT,[16] M. AM. SOC. C. E. (by letter).[16a]—This paper is an extremely interesting one as it places before the profession many facts that have not, heretofore, been available with respect to this extraordinary river. "China's Sorrow" is such an unusual stream, and the local conditions are so at variance with those that river engineers are accustomed to find elsewhere, that the description has many of the intriguing qualities of a Jules Verne novel. The quantities of sediment shown as passing the hydrometric station at Chinchang are so great that there should be outstanding evidences of buried structures throughout the alluvial plain, and Mr. Thorp has mentioned the case of a pavement buried to a depth of about 12 ft at Kaifeng, a few miles west of the 1851 break. The authors state that there is reason to believe that the serious erosion now taking place in Northwest China began at about the time of the Emperor Yü (2200 B. C.) but, even so, the depths of deposit in a period of 4 000 yr should have attracted more attention than is evidenced by the mention of a single case by Mr. Thorp.

Although the writer confesses a certain degree of skepticism with respect to the capacity of a river to carry sedimentary loads of the magnitude reported, the data have been accepted, for the sake of discussion, and an effort has been made to indicate some of the inevitable consequences that must follow from such a condition.

The slope of a river is one of its most important characteristics, and a profile is very useful in studying its habits. In Table 1 the authors give the necessary data and Fig. 36 has been prepared to show the slope from Shanchow (Mile 624) to the mouth. All elevations are referred to Taku Datum, the zero of which is 4.5 ft below mean sea level. As should be expected, when dealing with a stream as unstable as this one, the distances shown are somewhat different from those used by Mr. Freeman.[3] A number of stations not included in Table 1 have been approximately located by reference to the text or by scaling the distance to the nearest known point.

*Shanchow, Mile 624, to Chinchang, Mile 441.*—Between Shanchow and Mengtsin, Mile 503, the river flows through the last of its mountain gorges with numerous rock ledges and minor rapids; in this section it is an eroding stream carrying an enormous load of loess in suspension. A short distance above Mengtsin it enters its area of deposition and begins to meander from side to side of an alluvial valley in a bed composed of sandy material dropped by the river itself. A short distance above Chinchang (site of the Peiping-Hankow Railway Bridge) two tributaries, the Lo Ho and Chin Ho, enter the main stream. The importance of these branches as flood carriers was illustrated in 1935, when they added about 300 000 cu ft per sec to the July flood of that year. These tributaries drain an area covered largely by Tertiary deposits of red clay, and they do not carry as much sediment as the main stream at

[16] Col., U. S. A. (*Retired*), Washington, D. C.

[16a] Received by the Secretary February 1, 1939.

[3] "Flood Problems in China," by the late John R. Freeman, *Transactions*, Am. Soc. C. E., Vol. LXXXV (1922), p. 1405.

Shanchow. Moreover, the clay is finer than the loess and more easily transported. There are some strata of sand and gravel outcropping in the clay cliffs and these should add to the bed-load of the river. Certain maps, not published in the paper, indicate that the lower portion of the flood plain of the Chin Ho has been modified by a rise in the bed of the Yellow River.

In the rocky gorge above Mengtsin the average slope is about 4.6 ft per mile; and, in the alluvial section from Mengtsin to Chinchang, it is about 1.37 ft per mile. Throughout this alluvial section the bed is sandy and the river shifts from side to side of the flood plain; for the full length along the right bank the plain is bordered by the clay or loess hills; but on the left bank the dikes extend about 15 miles above Chinchang.

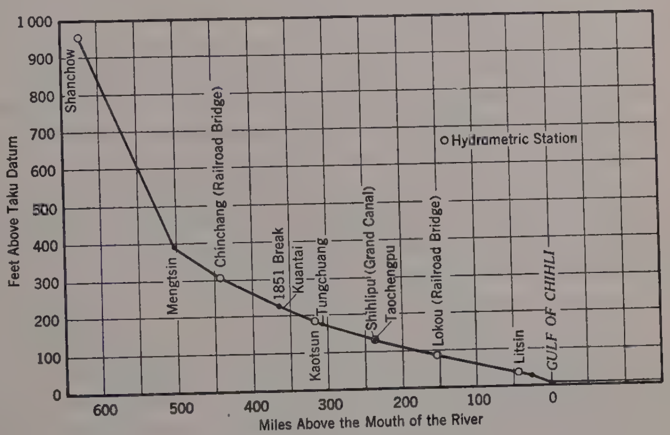


FIG. 36.—LOW-WATER PROFILE OF THE YELLOW RIVER

There is abundant evidence that the river is depositing huge quantities of sediment in the overflowed areas. The outflow from this section is measured at Chinchang and the portion of the inflow that passes Shanchow is also measured. No direct information is given with regard to the inflow from the Lo Ho and Chin Ho, and minor tributaries, but there is some indirect evidence that this amounts to several hundred million cubic yards per year.

Table 7 shows the monthly flow of sediment at Shanchow and at Chinchang for the years 1934–1935. During July, 1935, there was a flood in the tributaries amounting to more than 300 000 cu ft per sec, and during that month the outflow of sediment amounted to the inflow from above Shanchow, plus 158 000 000 cu yd. (For convenience in visualizing the extent of the flood-plain deposits it is well to recall that 1 000 000 cu yd may be taken, with an error of only 3%, as the volume of sediment required to bury 1 sq mile to a depth of 1 ft.) If, as seems probable, there were deposits during that month, the inflow from the tributaries must have been 158 000 000 cu yd plus the quantities deposited. During the 2-yr period there were nine other months in which the outflow was greater than the inflow from above Shanchow, the aggregate excess for the entire ten months having been 406 000 000 cu yd.

During these ten months the inflow of sediment from the tributaries was apparently 406 000 000 cu yd plus an indeterminate volume of deposits.

During the remaining fourteen months of the record the inflow of sediment from above Shanchow, alone, exceeded the outflow at Chinchang by 323 000 000 cu yd, and the deposits included this volume plus the entire inflow from the tributaries. The limited information available indicates that, for the years 1934–1935, the deposits on the overflowed land above Chinchang amounted to the huge total of more than 500 000 000 cu yd. For the flood plain below Chinchang, better information is, fortunately, available.

*Chinchang, Mile 441, to Tungchuang, Mile 307.*—The flow at Tungchuang will be assumed to be the same as at the hydrometric station at Kaotsun, a few miles up stream (see Fig. 29).

Throughout this 134-mile reach, the river flows along the back-bone of a ridge formed by the deposit of part of the sedimentary load. There are no tributaries; on the contrary, there is a continuous threat that a crevasse will form, permitting the wastage of part of the flood waters over the adjacent plain. In this section of the Yellow River the slope averages 0.94 ft per mile; the distance between dikes averages about 6 miles; the channel is constantly shifting its position between the dikes and there are many sandy islands and bars; there is a heavy bed-load; there are extensive sand deposits on the foreshores and on the adjacent plain; and, as shown by Mr. Freeman[5] all of the major changes in the course of the river have originated in this section.

At Mile 365 the great break of 1851 occurred and changed the course of the river from its former southeasterly direction to the Yellow Sea, to its present northeasterly course to the Gulf of Chihli. Mr. Freeman has presented a cross-section[17] about five miles above this break which shows that the dikes were about 6.25 miles apart; that the foreshores between the dikes and the channel had been built to a level about 30 ft above the adjacent plain; and that following the break the river had eroded deep into the deposits. In 1937, as explained by the authors, the foreshore lands, for a distance of about 25 miles above the site of the break, were still above the level of the highest floods. The river bed at this point is said to be rising at a rate of about 3 ft per century, but exact figures are not given. Down stream from the site of the break, toward Tungchuang, the natural and artificial levees have not yet developed to dimensions that may be considered reasonably secure. In the great flood of 1933 there were 35 breaks in a distance of 48 miles; in 1934 there was a break at Kuantai, only a few miles below the site of the 1851 break; and, in 1935 there was a break near Tungchuang, the lower end of the reach.

Table 8 shows that the quantity of sediment passing Chinchang during 1934 was 1 915 000 000 cu yd, and that only 1 168 000 000 cu yd passed Kaotsun; the difference, 747 000 000 cu yd, was deposited between the dikes or spread over the plain as a result of the Kuantai break. In 1935 the silt passing Chinchang amounted to 1 727 000 000 cu yd, of which 1 046 000 000 cu yd passed Kaotsun; the difference, 681 000 000 cu yd, was deposited between the dikes. In 1935 a considerable part of the sediment that had passed Kaotsun was subsequently spread over the plain as a result of the crevasse at Tungchuang.

[5] *Transactions*, Am. Soc. C. E., Vol. LXXXV (1922), Fig. 5, p. 1414.
[17] *Loc. cit.*, Fig. 12, p. 1421.

The profile of the river, the location of the various outlets since the beginning of the historical record, the descriptions of the shifting channels and numerous sand bars and islands in the upper reaches, the references to the relatively better defined channels in the lower reaches, the descriptions of the sand and gravel strata in the eroding Tertiary cliffs, the references to the bed-load and to the sand deposits on the foreshores and adjacent plain above the 1851 break, the data with respect to the silt loads at the various hydrometric stations, and the reports that the bed is rising—all of this evidence is in accord as pointing to the conclusion that the alluvial plain from Mengtsin to Tungchuang is now serving as a settling basin in which the coarsest part of the sedimentary load is deposited. The record of silt observations at the hydrometric stations and the statement that the river bed at the site of the 1851 break is rising at the rate of about 3 ft per century are in agreement as indicating that sediment is being deposited, but in notable disagreement as to the rate of such deposit.

The foregoing deductions should be checked by additional field surveys, and these should include mechanical analyses of the sedimentary deposits throughout the flood plain. Similar analyses should be made of the suspended load at each of the hydrometric stations; the sedimentary load should also be measured at one of the minor rapids above Mengtsin where the entire load will presumably be in suspension and therefore susceptible to being measured. The gage records at the Lokou (Mile 152) and Chinchang railroad bridges should be examined to see whether the low-water levels at those points are changing; and all available information with respect to changes in elevation of the river bed, the foreshores, or the adjacent plain should be assembled and studied.

*Tungchuang, Mile 307, to Litsin, Mile 42.*—The hydrometric station nearest to the mouth is at Litsin and the 265-mile reach from Tungchuang to that point will be considered as a unit. The Grand Canal crosses the river at Shihlipu (Mile 236) and, under certain conditions, water that has escaped through crevasses above that crossing is returned to the river at that point, after having been nearly completely clarified by sedimentation. From Tungchuang to the crossing the average distance between dikes is about 4 miles; below the canal the width is about 1.25 miles. Below the crossing the channel is more clearly defined than above that point. The average slope for the entire 265-mile reach is 0.54 ft per mile.

Table 8 shows that in 1934 the volume of silt entering the reach was 1 168 000 000 cu yd and the volume leaving at Litsin was 1 295 000 000 cu yd. In 1935 the crevasse near Tungchuang allowed the escape of an unknown, but very large, volume of water and sediment. The hydrometric station at Taochengpu, about Mile 236, showed an annual load of 514 000 000 cu yd as compared with 447 000 000 cu yd at Litsin. These values are not conclusive proof of either fill or scour between the dikes of this reach during the 2-yr period.

*Delta.*—In the discussion of plans for the control of the Yellow River, the authors state that the sediment to be discharged at the mouth may be as much as 815 000 000 cu yd per yr, a value far less than the quantity shown as having entered the diked channel at Chinchang during 1934 and 1935. Their discussion of the rate of advance of the delta into the Gulf of Chihli indicates that

no insuperable problem will be encountered at that point during the next century, but that very serious difficulties may arise within the next two centuries.

*General.*—When a project is under consideration for the control of a river flowing in an alluvial bed, between natural or artificial levees, the thought is often expressed that the stream should be so guided and controlled that the entire sedimentary load will be carried from the head of the valley to the sea without the deposition of any part en route. Assuming, without demonstration, that this can be accomplished in the flat sections near the mouth, the proponents then look at the steeper slopes in the upper reaches of the area of deposition and propose, further, that the same methods of control be applied to the reaches of relatively steep slope. They argue that this should cause erosion and that the river should cut its channel deep into the friable deposits, and thus lower the high-water surface to a level below that of the surrounding plain.

The Emperor Yü (2200 B.C.) is alleged to have controlled the Yellow River by causing the flood waters to flow to the sea at levels below those of the surrounding plain. Unfortunately for humanity, this tradition must be classed as a myth or the method used 4 000 yr ago must be cataloged as one of the lost arts. In recent years, when it is proposed to make the Yellow River dig deep, the method advocated calls for a reduction in width and a debate starts as to whether this contraction will result in a lowering of the bed or whether it will cause it to rise at a faster rate than at present. Those claiming that the bed will be lowered base their arguments on the higher velocities that will be found in the narrowed channels, and the observed scour that has been noted in certain short reaches where the channel now has the width proposed. Those believing that the bed will be raised hold to the opinion that the highest velocities obtainable in practice will be lower than those necessary to carry the entire sedimentary load to the sea, and that the reduction in the area over which the deposits will be laid down will increase the depth of the annual accretions.

In nature, an alluvial river carrying a heavy bottom load builds for itself a wide bed with a slope sufficient to create an adequate bottom velocity. In discussing the effect of structures designed to reduce the width of such a stream, the increase in depth and consequent increase in mean velocity is habitually stressed. There are other factors of equal importance, however, although frequently they are left unmentioned. Exact data are not always available and, for a mental picture of the case, it may at times be advisable to depart from strictly scientific accuracy and resort to the use of rough estimates that can, at least approximately, be checked by field observation.

From the surface to within about 1 ft of the bottom, the suspended load and velocity of the river current can be measured with reasonable precision; by extrapolation the figures for a point 0.5 ft above the bottom can be closely approximated. For purposes of discussion the velocity 0.5 ft above the bottom will be used as one of the indices. At a point about 0.2 ft above the bottom, or possibly less, a movement of sand by the saltation process comes into action, the character of the load is greatly altered, the load is much increased, and the velocity of the water is reduced. No practicable method of measuring the

velocity of the water and the extent of the sand movement within 0.2 ft of the bottom has been devised.

The sand, traveling by saltation, together with the much smaller quantity that moves by rolling or sliding, will be referred to as the bed-load. In the absence of information as to the magnitude of this load in the Yellow River, a very rough estimate will be made in the hope that it will be better than nothing. Based on experiments by the late Grove Karl Gilbert,[18] the bed-load in the Yellow River when the mean velocity is 5 ft per sec, and the velocity 0.5 ft above the bottom is 3.0 ft per sec, will be assumed to be 300 grams per sec per ft width of channel; this is equivalent to 20 cu yd per day per ft of width.

If the depth of a stream is 10 ft, its mean velocity 5 ft per sec, the suspended load 6%, and if 1 cu yd of deposit weighs 2 800 lb, the suspended load per 1 ft of width will amount to 5 800 cu yd per day. These rough estimates indicate that if one-third of 1% of the suspended load in the gorge above Mengtsin consists of sand too coarse to be carried in suspension through the alluvial channel, the capacity of the river to carry bed-load may be fully taxed. The profile of the river and the descriptions of its characteristics indicate that its bed-load capacity is over-taxed. (On the Missouri and Colorado rivers in the United States, a rough rule is that sand too coarse to pass through a 200-mesh sieve will be actual or potential bed-load material. The slope of the Colorado River is about 1.5 ft per mile, and that of the Missouri River about 0.8 ft per mile. Mr. Freeman stated[3] that more than 99% of the Yellow River sediment would pass through a 200-mesh sieve.)

It is not easy for a river carrying a heavy bed-load to dig vertically downward. As an alluvial plain is built up the natural process tends to pave the bed with the coarsest material available; any subsequent attempt to dig vertically downward results in the formation of a more and more resistant surface. On the other hand, the natural levees are composed of material carried in suspension until the velocities along the foreshores have sufficiently slackened to permit its deposit, the coarsest part of the suspended load being normally deposited nearest to the channel. These bank deposits are conspicuously lacking in cohesive qualities and are very vulnerable to attack by undermining or sapping. In expanding in a lateral direction the current need only loosen part of the material to be excavated; the remainder is loosened by the caving process. Material lifted from the bottom by the current is so coarse that much of it must be carried away by the slow process of bed traction, whereas material loosened from the banks, for the greater part, can be borne away readily in suspension. When a river must enlarge its section by excavation it is often easier for it to accomplish its purpose by digging laterally than by digging vertically downward.

If $q$ is the bed-load capacity of a river per 1-ft width and $B$ is the width in feet, $q\,B$ is the bed-load capacity of the entire stream. The unit capacity, $q$, can be increased by an increase in velocity, and this in turn can be secured by a reduction in width. However, if the reduction in width is at a faster rate than the increase in unit capacity, the bed-load capacity of the entire river will be reduced instead of increased.

[18] "The Transportation of Débris by Running Water," *Professional Paper No. 86*, U. S. Geological Survey, 1914.

The foregoing considerations create a doubt as to whether it would be possible to increase the bed-load capacity of the Yellow River by reducing its width, or that it would be possible to induce the stream to dig vertically downward to a measurable extent; but if these objectives could be attained, the present flattening of the river as it flows downward through its alluvial section would introduce further difficulties. From Mengtsin to Chinchang, a distance of 62 miles, the present slope averages 1.37 ft per mile, and the Yellow River is unable, nevertheless, to carry its full sedimentary burden through that reach. If the use of contraction works and revetments were to make it possible for the stream to carry its full load through the aforementioned section, the problem would arise as to how the stream could be trained to carry this load, larger than the present one, through the next reach to Tungchuang, a distance of 134 miles, with a present slope of 0.94 ft per mile. If the problems of both of the aforementioned sections were solved there would remain the task of training the river to carry this full load through the remaining 307 miles to the sea, the present slope being only 0.54 ft per mile.

In their discussion of a possible project, the authors state that the regulated channel should be competent to carry to the sea an annual load of 815 000 000 cu yd. The volume entering the diked channel at Chinchang is stated as 1 727 000 000 cu yd in 1935 and 1 915 000 000 cu yd in 1934; and, the quantity carried by the much greater flood of 1933 is not known.

It would appear that the project for the control of the Yellow River for the next century should include provision for the deposit of a huge volume of the coarsest sediment, in an area above Tungchuang, diked off and reserved for that primary purpose. Below Tungchuang the project might provide for carrying the partly desilted waters, as at present, with only moderate deposits on such foreshores as may be left open between the dikes and the edges of the channel.

In selecting the proper area to carry the flood waters as distinguished from the sediment, the authors are on firm ground when they propose to give the regulated river a width that will carry the floods without making allowance for beneficial scour. The writer is not convinced that, if detention basins of the type suggested be built, it can be safely assumed that beneficial scour will result and that the width of the regulated channel can be reduced correspondingly.

Corrections for *Transactions:* Page 1921, Line 10 below "Synopsis," change "1928" to "1919"; Page 1928, Line 14, change sentence to read "Likewise was its control * * * effected by a single line of strong dikes."

AMERICAN SOCIETY OF CIVIL ENGINEERS

Founded November 5, 1852

DISCUSSIONS

# TRAFFIC PROBLEMS IN METROPOLITAN AREAS

**Discussion**

BY MESSRS. GEORGE H. HERROLD, AND THOMAS BUCKLEY

GEORGE H. HERROLD,[3] M. AM. SOC. C. E. (by letter).[3a]—Observations and analysis of accident reports lead the writer to believe that the accident is most likely to occur where movements are simple and clearly evident to drivers and pedestrians, and that where the movements are complex and confusing, and drivers and pedestrians do not fully understand the situation, they proceed cautiously and safely. This would indicate that the cause of many accidents is in the driver. The careless driver does his damage where movements are simple and clearly defined. Careful drivers are becoming better drivers. They are constantly improving their technique, but the accident-prone driver presents a problem with which the careful driver cannot cope.

The writer has had an analysis made of several hundred accident records which were kept on forms presumably approved by the National Safety Council as filled out by police officers. They show where, when, and how the accident occurred; but it takes a keen analysis to determine the cause of an accident from these reports, because the "how" is not the "cause." In fact, it requires a trained investigator, of which there are very few. The report shows that "Vehicle No. 1, going north, collided with Vehicle No. 2, going west, at the intersection of 'X' Street and 'G' Avenue; (date), 10 A.M.; daylight; pavement dry; no defects; clear weather; drivers both males; not under influence of liquor; brakes O. K.; no car defects apparent: Driver No. 1 wore a brace on right foot; no signal or warning signs at the intersection; both drivers going straight through; and both had a license." The report shows where, when, and "how" the accident occurred (Driver No. 1 collided with Driver No. 2) but the cause of the accident can only be surmised. Probably both drivers were stubborn and would not "give an inch."

Accident reports are valuable; but the "vital point" (that is, human reactions at time of accident) is difficult to establish. An analysis of these reports by trained investigators is the real solution in helping traffic through danger

NOTE.—This paper by Earl J. Reeder, Esq., was published in December, 1938, *Proceedings*. This discussion is printed in *Proceedings* in order that the views expressed may be brought before all members for further discussion of the paper.

[3] Planning Engr., City Planning Board, St. Paul, Minn.

[3a] Received by the Secretary January 9, 1939.

spots. The railroads are correct in requiring a long course of training for a locomotive driver.

Records of "Stop Sign" violations and repeat violations by the same cars indicate that people refuse to stop where there is no vehicle in sight (see Table 1). The suggestion of another sign to take the place of the stop sign in many places, to read "slow to 10 miles" or slow to the computed critical speed is an excellent suggestion.

TABLE 1.—OBSERVANCE OF STOP SIGNS BY 58 165 DRIVERS AT ELEVEN INTERSECTIONS IN ST. PAUL, MINN.
(7:00 A.M. to 7:00 P.M.)

| Description | (a) CAME TO FULL STOP | | | (b) SHIFTED GEARS BUT DID NOT STOP | | | (c) SLOWED BUT DID NOT STOP | | | (d) WENT STRAIGHT THROUGH | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | 1934 | 1935 | 1936 | 1934 | 1935 | 1936 | 1934 | 1935 | 1936 | 1934 | 1935 | 1936 |
| Number of vehicles....... | 24 827 | | | 25 764 | | | 5 249 | | | 2 325 | | |
| Percentage of vehicles..... | 44.2 | 44.5 | 42.7 | 36.8 | 42.2 | 44.3 | 17.1 | 10.1 | 9 | 1.9 | 3.2 | 4 |

There is an unfortunate lag between increasing traffic congestion and the merchant's acknowledgment that he is losing business. Blight affects mercantile streets in many cities because of this lag. If the problem had been analyzed in the beginning a suitable by-pass could have been arranged and thus saved or delayed another blighted area. Proper routing through cities, and education as to the purpose of these routings, are among the essential necessities.

Every one knows that too high speeds are used by drivers in business districts. When a business man walks from his office to the bank he could save considerable time if he ran all the way; but he does not run. In fact, it has never occurred to him that he could save time by running. He walks in a dignified manner from one place to the other. The automobilist must also learn to move in a dignified manner when on congested business streets.

*Providing for Loading and Parking.*—The purpose of parking a car is to enable the driver to leave it, while attending to one errand or another. In most cities where an off-street parking space has been provided it is only about half in use; that is, twice the number of cars could probably be cared for in existing parking lots and garages when the people have been educated to make use of them. In St. Paul, Minn., the off-street storage capacity in the central business district is for 9 560 cars. The used capacity is 5 655 cars. In addition, parking space provided by utility companies and other private concerns, on their own grounds, for employees' cars, totals 1 630. Much the same situation occurs elsewhere. City officials have not yet decided whether it is a part of their business to provide parking spaces off the street, although a number are doing so on city-owned or tax-delinquent lands.

In St. Paul, with 90-min parking in the central business district, surveys developed 82% legal parking and 18% illegal parking, with 62% of all legal parking remaining at the curb less than 30 min. Based on these facts the parking ordinance was changed to provide 30-min and 60-min parking as more nearly in line with actual needs. In the 1937 survey it was found that 56% of the cars are legally parked and 44% illegally parked. The percentage of illegal parking increased, of course, with shortening of parking time and with the increased use of cars. The report of the National Association of Building Owners and Managers on parking meters, dated October, 1938, leaves no doubt that parking meters will cure this situation.

It is possible that the eventual solution of the parking and loading problem lies in condemning older buildings and land in the retail district, clearing the ground, and using it for parking of cars and "off-street loading," assessing the cost of such operation to the surrounding property owners. This would provide a free parking space with a limit of 1 or 2 hr to insure a logical turnover and a cash charge for any parking over-time. Such an area for off-street parking and loading should be planned to fit into the mosaic of the retail and office building district.

*Mass Transportation.*—An indication of the traffic picture of a city is the morning flow taking people to work. In 1924, 541 street cars brought into the St. Paul business district, in the morning, 31 000 passengers, whereas 9 250 automobiles brought in 14 677 passengers. In 1938, 419 street cars brought in 17 000 passengers; 71 buses, 763 passengers; and 18 100 autos and other vehicles, 29 100 passengers. In addition there were 7 200 pedestrians walking to work in 1938, a situation which was not considered important enough to check in 1924. Street space in St. Paul has been increased quite materially since 1924 but not in any ratio comparable to this increase in volume of traffic or use of street space—and this is true of many cities. Only by providing mass transportation that is more comfortable and attractive than a private automobile (such as the electric trolley bus) and limiting the use of private cars by those who do not need them downtown during the day (by enforcing drastic "no parking" regulations or parking meters, as is now proving successful in fifty cities) can this situation be altered for the better.

THOMAS BUCKLEY,[4] M. AM. SOC. C. E. (by letter).[4a]—A concise and interesting arrangement of traffic problems is presented in this paper. Sub-headings are neatly phrased and carefully chosen to include essential phases. They are in a form worthy of attention because they are adaptable to use outside the boundaries of purely technical review; and they lose none of their effectiveness if restricted solely to observation and discussion by engineers. Descriptive captions such as those in the paper prompt a reminder that the most urgent problem of the traffic administrator is to catch and hold public attention and to engender a respect for the profound importance of the personal factors in all traffic problems. After all, traffic control is outstanding among all technical fields as a procedure wherein the primary fundamental problem is to find

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[4a] Received by the Secretary January 10, 1939.

effective ways and means of educating the public at large in human (and humane) behavior; or, in other words, to a full realization, and to the point of permanent, co-operative acceptance of their individual personal responsibilities as joint users and sharers of streets and highways. The extent of this problem is discernible in the following quotation:[5]

"By what method can we make people realize that it isn't real fun to take chances with a motor car? What persuasions will be strong enough to convince the motorist that he should lose a minute and save a life? What means can be used to convince everybody that they should not drive after drinking?

"How can taxpayers be persuaded that a drivers' training program in the high school is at least as important as a luxurious swimming pool? What kind of propaganda will it take to make drivers as courteous at the wheel as they are in the drawing room?"

Through the centuries the "human" characteristics of carelessness, thoughtlessness, heedlessness, selfishness, stubbornness, rudeness, and ruthlessness have become deeply imbedded in Man's nature. Unfortunately for his fellowmen, and particularly for traffic engineers, he elects to use streets and highways as places in which to display the mental perversities mentioned. Solution of the problems of self-control will never keep apace with the progress made in mechanical and design controls.

The stressing of the time (speed) and safety factors by the author in his Synopsis suggests a second comment. This is that three utilitarian tests of a good highway should be emphasized more frequently as such. It is also a reminder that the third one of these tests seldom receives consideration commensurate with its importance in traffic problems. The three tests in mind are those of speed (time), safety, and comfort. The first two of these tests are under discussion publicly more or less continuously but the third (comfort) is seldom mentioned. This circumstance is probably due to the general acceptance of "comfort" as being covered by the "safety."

A well-designed highway is one that permits reasonable speed with minimum hazards and, in addition, assures maximum comfort to all its pedestrians and vehicular users—comfort not only in the sense of that "bodily ease," which is afforded by efficient design and construction, but including, as well, that comfort resulting from "mental ease," which can only be realized when travel upon public highways is guided and protected by effective controls. Mental dread or fear of highway hazards is particularly strong among young and aged pedestrians. It is not entirely absent in the minds of many automobile operators. Its presence under test may induce the fatal mental panic which so often contributes to the production of highway accidents. Tests for highway comfort must also take into consideration those lesser forms of highway friction, such as the crowdings, jostling, blocking, bumping, etc., constantly occurring during business hours on the sidewalks and in the roadways of the principal streets in main, wholesale, and retail centers of urban areas. Although the great majority of such happenings do not result in serious property or personal harm, they add materially to the mental stress and strain of daily life in large cities. These conditions are plainly evident at the crossings of

[5] See editorial: "Stop and Go," *Public Safety*, December, 1938.

countless highway intersections. The problem of comfort to pedestrian and rider at the junctions of main streets will never be completely solved; but, with such lessons in mind, new streets and highways should be designed and controlled, as far as it is economically possible, so that pedestrian and vehicular users may derive and share maximum enjoyment of them. To accomplish this end, highway discomfort must be reduced as much as controlling circumstances permit. A gospel of "friendly roads" with a mutual sharing of their privileges must be taught.

The author's consideration of the traffic situation in metropolitan areas is naturally concentrated on problems common to the urban or improved sections of such areas. Difficulties encountered in urban traffic problems are generally increased and their solutions made impossible of completion because of original planning defects or deficiencies. Owing to the nature of urban areas, controls such as signals, signs, markings, etc., with occasional islands, zones, channelizations, etc., constitute the common ultimate methods of solving the major group of traffic problems. These measures cannot entirely overcome the handicaps and hazards to traffic resulting from structural faults such as poor alignment, bad grades, lack of visibility, insufficient lighting, inadequate cross-section space, weak intersection layouts, etc. Few towns and cities are free of problems arising from such conditions. The lessons derived from traffic problems arising in urban sections of metropolitan areas should be applied advantageously not only in the planning and improving of new streets, but in replanning, wherever possible, all existing planned streets that are not developed as yet to a point which precludes corrections.

Metropolitan areas generally include, in addition to their built-up and partly built-upon sections, areas that are wholly unimproved. In some jurisdictions, these unimproved districts may actually include lands that are in two important stages of evolution: One class comprises planned sections (that is, areas laid out into streets and highways upon an official map or city plan, but not physically improved) and the second class embraces areas that are at present unplanned but are "ripening" rapidly for planning and development. What future traffic problems are hidden in such areas? Can proper precautions be taken so that these new sections will respond to a higher degree of traffic control with fewer problems than old urban areas? The municipal engineer and planner are deeply interested in these questions and from their viewpoint a discussion of traffic problems in metropolitan areas must also include the potential problems that may exist within already planned but unimproved sections, or which may happen by reason of incomplete future planning.

In his "Conclusion" the author rightly declares that "traffic planning is distinctly an engineering function." As such, it must acquire a vision which reaches out beyond the problems and necessities of traffic control installations on existing highways. It must begin with, and have a definite place in, the original planning. It must undertake to assure that areas already planned for unimproved districts are redesigned, if necessary, to meet reasonable traffic requirements and that they are adequately adapted for rational controls. The planner's objective should be to so plan or replan, and to so guide trends in

development, that future traffic problems in the areas affected will approach an irreducible minimum in so far as damaging highway friction is concerned.

It is the responsibility of the engineer to develop and apply sound practice in the design and construction of streets and highways. To accomplish this objective, he has been meeting the requirements of speed, safety, and comfort for years by endeavoring to provide highways with dry foundations, solid and durable paving structures, safe grades, visible alignment, and even surface. To these qualities, he must now add that of traffic control, and co-operate with the planner to accomplish this end.

Modern highway use has produced new standards in the factors of volumes, intensities, loads, speeds, etc., which in turn have established new requirements with respect to the structural elements of highways—alignment, grades, cross-section design (including channelization), paving structures, curbing, drainage, fencing, railings, etc. All of these elements are directly connected with, and are an intimate part of, the problems of traffic control. Not only is sound engineering technique in design essential in minimizing traffic problems, but first-class construction and efficient maintenance are equally important. Worn-out or otherwise defective paving on certain streets and highways can disarrange the best of traffic controls completely and thus create a number of new local traffic problems. The design, construction, and maintenance of highways can be carried to any degree of efficiency required. Safeguards for traffic can be provided that will function to a high state of effectiveness. The difficulty encountered in building and equipping such highways is a very material one. Generally there are economic limitations to the extent to which such improvements are justified; and, in concluding this discussion, the writer raises another basic question in traffic control: "How can finances be obtained for furnishing, satisfactorily, the essentials—the means—necessary to complete the solutions of the many traffic problems after the ways of solving them have been fully determined? The problem of financing the solution of traffic problems deserves considerably more attention.

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DISCUSSIONS

# EARTHQUAKES AND STRUCTURES

**Discussion**

## BY MESSRS. HOMER M. HADLEY, AND R. McC. BEANFIELD

HOMER M. HADLEY,[5] ASSOC. M. AM. SOC. C. E. (by letter).[5a]—Any designer undertaking to design an entire structure or an individual part thereof is confronted, at the very outset of his labors, with the question of loading. To what loads will his structure be subjected; what is their magnitude; their direction; their point of application; their duration; their periodicity, etc.?

It is refreshing, therefore, to read the authors' admirable statement of the nature of the movements that cause and create the earthquake loadings (see paragraph preceding "Earthquakes and Their Effects: Relation to Structures" in Part I).

This is a plain, true description of the earthquake movements of the ground which every seismographic record ever taken clearly attests. A veritable chaos of confused, conflicting movements follows upon the primary initial disturbance, which may occur—who knows when, or how? The authors state (see "The Problem"): "It would appear to be beyond the ability of any mathematician to develop formulas that would take accurate account of the extremely variable nature of the factors involved." Assuredly this is so.

To the writer it has always seemed a waste of effort to use methods of design more refined than basic conditions warrant, and the clear presentation, in this paper, of the utter ignorance there is of the earthquake loading, prompts the suggestion that structural engineers may well recognize the limitations of their knowledge. When the magnitude and direction of the loads is unknown, what choice is there between a dynamical or statical approach to a solution as arbitrary as, and no more valuable than, the good guesses called assumptions made at the outset?

In the short insert appendix to his book "Earthquake Damage and Earthquake Insurance" the late John R. Freeman, Past-President and Hon. M. Am. Soc. C. E., gives "Some After-Thoughts by J. R. F. and a Few Notes from the

NOTE.—This paper by Leander M. Hoskins, Esq., and John D. Galloway, M. Am. Soc. C. E., was published in December, 1938, *Proceedings*. This discussion is printed in *Proceedings*, in order that the views expressed may be brought before all members for further discussion of the paper.

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[5a] Received by the Secretary January 3, 1939.

Suyehiro Lectures."[6] The views summarized therein are most interesting and valuable, particularly the concluding paragraphs containing the suggestion—originating with Professor Suyehiro, a former naval architect—that quite as sea-going ships are designed successfully by assuming them to be riding on "standard" waves without knowing much about the actual waves and quartering seas which they encounter, so similarly one may, arbitrarily and successfully, design buildings against earthquakes by assuming a conventionalized loading. The fact that buildings in Tokyo designed under "the $0.1g$ basis" resisted the 1923 earthquake fairly well is called "a datum more valuable than any other arguments."

It would seem desirable, therefore, to adopt as design procedure nothing more complicated or involved than the assumption of horizontal forces acting in any direction upon the structure and applied at its base. Logically and properly, the magnitude of these forces is a function of the weight of the structure and its loads: $0.10g$, $0.08g$, or whatever judgment dictates. The damping effect which the height of very tall buildings has upon stresses may be recognized by using a smaller coefficient of $g$; or, as Mr. Freeman suggests, by model experimentation on a shaking table and special treatment. The important consequence of a design developed on this basis is not that computed stresses will be those actually developed in an earthquake, but that reversals of stress will be provided for, connections will be connections, anchorages will be adequate. All the sorry structural spectacles that have characterized past earthquakes have not been consequent upon the failure to use this or that cunning formula or method of design. They have been due to failure to provide proper resistance to horizontal forces even roughly approximate to those occurring.

Some additional comment may be offered on certain minor points. The authors state ("Earthquakes and Their Effects: Relation to Structures," in Part I) that since the forces generated in a structure subjected to earth movements are measured by the mass or weight of the building it follows that "the use of light materials in a structure is the best and that heavy, inelastic materials should be avoided as far as possible." Actually what follows is that with light materials the forces generated in a structure are of less magnitude than those developed with heavy materials. This is not, however, a criterion by which to assess the relative worth of the two constructions. Certainly on the basis of cost to the owner it may well prove more economical to specify the heavier material; and since the earthquake behavior of a structure does not result from the inert, lifeless materials of which it is built, but from the manner in which these materials are shaped and assembled and are combined together, it is not apparent that the weight of the construction has any appreciable bearing on the matter at issue.

Inasmuch as buildings "with wings at right angles" may be so built and connected as to sustain no earthquake damage, it is suggested that the paragraph dealing with the form of the structure be modified so that it indicates the hazards and dangers of this shape rather than pronounces inevitable doom. There is an almost equal hazard and danger in a square or rectangular building

[6] *Proceedings*, Am. Soc. C. E., May, 1932, Part 2.

wherein the resistance to horizontal forces is most unequally and unsymmetrically distributed with respect to the plan of the building.

Regarding "visible surface waves" in the ground, to the writer (who has never chanced to see them), it has always seemed quite strange that they should occur without causing a general and extensive shattering of concrete sidewalks, curbs, etc. He has seen the rupturing of concrete pavements at fissures; but the waves, if any, apparently do no damage. In the fall of 1923 in Tokyo, a friend of the writer was in the Imperial Hotel when one of the countless after-shocks occurred. This was a very severe one, however, and it caused considerable alarm. Telling about it in the calm of the evening, he said that he had rushed to the window of his room, and looking out, it seemed as if the long side-wall of the hotel was "waving like a flag in the wind." He did not believe that the wall did wave, but he asserted that it seemed to. Bricks and concrete being intact, the phenomenon undoubtedly was a personal one—"in my mind's eye, Horatio."

R. McC. Beanfield,[7] M. Am. Soc. C. E. (by letter).[7a]—This paper is helpful and thought-provoking and will tend no doubt to stimulate interest in this important subject, which has been somewhat neglected by the profession in general. The paper contains much reiteration of known data; Part II, covering stresses and dynamic theory, treats the subject more from an academic point of view, and fails to offer any practical solutions. It also fails to include much advanced development in the art of aseismic design subsequent to 1928. The fact is that many engineers are mindful of the inadequacy or incompleteness of the theory and practice relative to aseismic design. The present state of the art is rather empirical and is based to a large extent on opinions and conjectures which are not substantiated by fact. Engineering is essentially a constructive art, and builds from known facts.

The following fundamental facts have been developed by scientific research, structural dynamics, and experimental investigation:

(1) When a vibratory motion is applied to the support of a system, the system itself will generally begin to vibrate. In the early stages of its vibration the motion of the system does not bear a simple relation to that given to the support. During this interval, the resulting motion is made up of two parts: "Free," or transient, vibrations; and "forced," or steady-state, vibrations. If the system is initially at rest when the disturbing motion begins, free vibration will always exist during the early part of the motion. These transient vibrations may cause the displacements of the structural system to be more than twice as great as those that will eventually exist in the steady state.

(2) Earthquake motion is not simple motion even if one dimension is considered. It consists of elastic impacts followed by free vibration and forced vibration with all their transients superimposed. The period, or apparent period, varies throughout the duration of the earthquake although in a number of actual, strong-motion seismographs certain periods dominate.

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[7a] Received by the Secretary February 6, 1939.

(3) Seismic ground vibrations (in so far as records to date reveal them) are not harmonic; on the other hand, the vibration of elastic yielding structures, caused by the disturbing force of seismic impulses, is harmonic, which fortunately tends to simplify the dynamic problem.

(4) The presence of transients may cause very high stresses within the duration of the first cycle or the first two cycles of the disturbing motion, which indicates that earthquakes do not require sustained vibrations in order to cause dangerous stresses in structures. Displacements of an oscillating structure are directly proportional to the ground amplitude. Therefore, the structure may be out of resonance with the ground and yet be over-stressed because of the large amplitude of the ground motion. Earthquake periods need not be in direct resonance with the natural periods of a structure in order to produce dangerous dynamical stress effects.

(5) The application of a fictitious or supposititious static force, derived from the conventional formula $H = \frac{W\,\alpha}{g}$ (wherein a maximum acceleration of the ground is assumed to be the same through all parts of the structure above the ground), may be used to obtain a practical solution only for relatively stiff structures such as one-story or two-story block-type buildings of average height. Its use as a general formula for aseismic design in elastic yielding structures such as water-towers, chimneys, multi-story frames, etc., is unscientific and may result in a false sense of security and structural stability.

(6) The conventional measure of the intensity of the earthquake known as the seismic factor is the ratio of the greatest horizontal acceleration of the ground to the acceleration due to gravity. It is a well-established fact that the only proper method of judging earthquake intensity is by the kinetic energy method. It can be demonstrated by structural dynamics that the kinetic energy decreases as the frequency increases, or as the period shortens, and that the amplitude of motion decreases very much faster than the frequency increases. These results prove that the acceleration used in the conventional statical formulas is not a constant and loses its usefulness as a measure of the intensity of the earthquake. It is irrational to assume a single constant value of the intensity ratio or seismic factor independent of the periods.

(7) The natural period of a structure is a criterion of its rigidity.

(8) Dampening has little effect in reducing the maximum amplitude of the transient vibration and possible dangerous stresses during the first few cycles of vibration. The stiffening effects due to the friction of joints and of interior secondary partitions, such as plaster, wood, or tile, should not be given any value in computing displacements of the structural frame.

(9) Elastic yielding structures, particularly the more flexible type, do not always vibrate only in their fundamental modes. As a matter of fact the motion of the ground causes harmonics of frequencies higher than the fundamental, and there is always the probability that one of these harmonics, usually within the first five modes, is close enough to the ground oscillation to cause dangerous dynamic forces. In the more elastic types of structures, such as towers, the maximum natural period is limited by conditions of buckling.[8]

[8] "Die Erbebensicherheit von Bauwerken," by Dr. R. Briske.

The statement in the paper (see heading "Dynamic Theory as a Guide to Practical Design"), that the solution on the assumption of rigidity is simple and definite, may be questioned. The structural designs, in which a fictitious horizontal static force is used, do not comprise a simple problem. As a matter of fact, in order to obtain similar rigidities throughout the structure, considerable ingenuity and a good practical knowledge of the application of the elastic theory are required.[9]

The "Fundamental Practical Rule," which the authors infer can be used with confidence (see "Dynamical Theory as a Guide to Practical Design"), does not apply to all types of structures. It should be understood definitely, and emphasized, that this rule applies only to inherently rigid structures, such as one-story or two-story buildings that have natural periods of exceedingly high frequencies.

Attention is called to a common error in the use of the fixed-force method wherein the horizontal force or supposititious shear (used, for example, at each floor level in a structural frame) is based on a constant value of the maximum ground acceleration, usually 0.1 $g$, whereas the actual acceleration at each floor level varies and increases from the ground up. It is obvious that, as the deflection increases upward, so does the acceleration.

All structures that are elastic yielding should be designed by some method of dynamic analysis. In this connection the writer suggests the following method.[10]

(a) Compute the natural period of the structure by determining the maximum deflection or displacement due to horizontal disturbing forces based on the seismic factor of 0.1 $g$. If the maximum deflection of an elastic structure within its elastic limit is known, then, by structural dynamics, the maximum period, acceleration, and dynamical shears can be obtained. The maximum statical deflection of an elastic structure is also approximately the maximum amplitude of its transient vibrations.[11]

(b) By the use of the magnification factor $B$, the maximum stresses, which should not exceed the elastic limit, can be determined by the formula

$$B = \frac{1}{1 - \frac{T_0^2}{T^2}} \qquad \text{(51)}$$

in which $T_0$ = natural period; $T$ = period of disturbing forces; and, $B$ = index of the increase in deflections, bending moments, and stresses due to forced vibrations.

(c) The natural period of a structure ascertained by the foregoing method should not exceed 0.5 sec.

It is an established fact that the dynamic responses of structures to earthquakes of high frequencies or short periods do not have large displacements of

[9] "Earthquake-Resisting Construction," by Dr. T. Naito.

[10] "Vibrations and Their Effects on Structures," by R. McC. Beanfield, Applied Science Div., Local Section, A. S. M. E., Los Angeles, Calif., April 25, 1934.

[11] "Vibrational Problems in Engineering," by S. Timoshenko, Rayleigh's Energy Method, p. 63.

amplitude. Therefore, structures with high frequencies will not be dangerously affected, and it is also true that the duration of seismic frequencies is usually short.

Attention should be called to the fact that application of the various formulas based on the dynamic analysis of a thin rod of constant cross-section, even for comparative purposes, in the design of structures, may lead to erroneous and dangerous results. In a number of the flexible structures such as chimneys and towers, for example, the mass of these structures is not usually uniform, the moments of inertia vary, and the deformation is caused more by shear than by bending moments. For instance, the deformation of structural frames is due more to shear which completely changes the formula of the thin rod concept. Therefore, it is not applicable to such structures. "Formulas are the anesthetics of thought, not its stimulants."

*What Materials Should Be Used?*—Invariably, buildings are described as brick, concrete, or steel-frame structures. It must be emphasized that it is not the materials but the design that is of foremost importance. There are certain requisites for materials, and certain materials fit these particular requirements better than others. As to this suitability, there has been much ignorant propaganda and some deliberate misrepresentation. If the engineer will prepare a design and compute the stresses, he will see for himself which materials are required for a given place in the structure.